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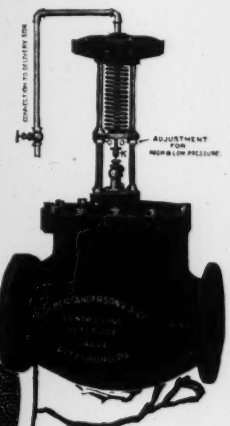

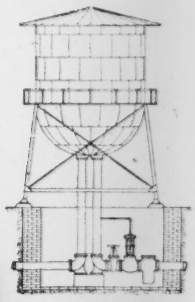

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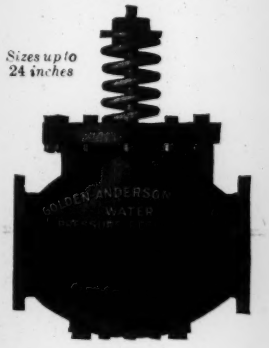

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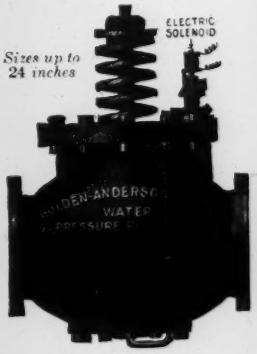

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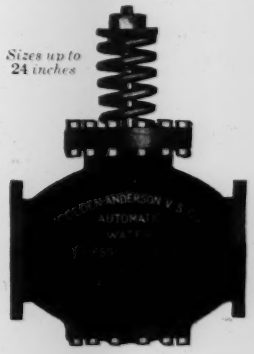

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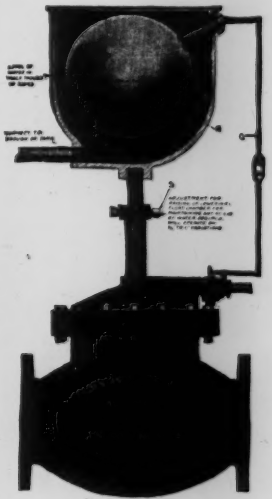

Golden-Anderson Valve Specialty Co.
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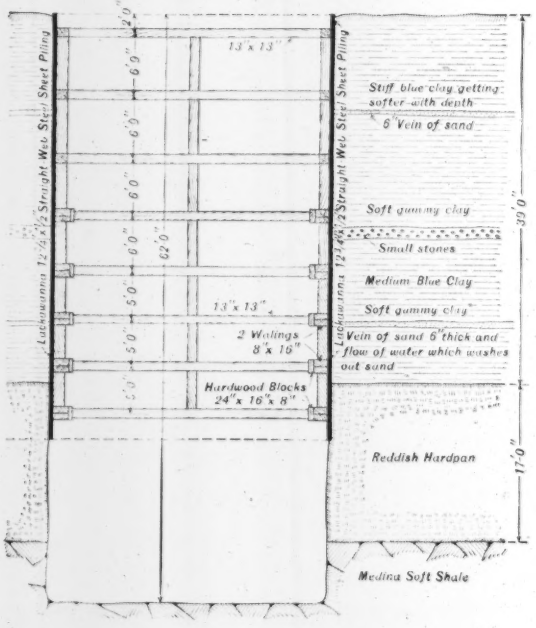
Lackawanna Steel Sheet Piling Cofferdam In Which The Breast Wall For Lock No. 2 Of The Welland Ship Canal Was Built.

The steel sheet piling here, driven in 45-foot lengths, in accordance with the cofferdam design shown in our last week's advertisement reached refusal in hardpan at 38 feet, and was left at this position until after excavation to the bottom ends of the sheet piling sections, when the wedges between each piling section and the timber waling were knocked out from one pile at a time and the pile was driven into hardpan at 45-foot depth This was accomplished without difficulty, the wedges replaced and succeeding piles treated in the same manner, after which excavation proceeded to the bottom. An 8,000-lb. steam hammer drove about 20 of the steel sheet piling sections in a day.



The interlocked joints of the steel sheet piling became practically watertight. Although the old Welland Canal ran within 20 feet of one side of the cofferdam, water entered only under the toe of the piling wall and was easily kept down with a 5-in. pump.

The accuracy to which Lackawanna Steel Sheet Piling can be driven is indicated in the above picture and is further indicated by the fact that the closure in the steel sheet piling wall was made without the slightest difficulty.

Other pictures of this work after the Lackawanna Steel Sheet Piling had served its purpose will be shown in our advertisement next week.

In the meantime you would be well repaid by reading our book—"Lackawanna Steel Sheet Piling" which tells about other important cofferdam work and will be sent free to managing and contracting engineers.

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General Sales Office and Works: Lackawanna, N. Y.

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Engineering News

VOLUME 75 JUNE 22, 1916 NUMBER 25

Emergency Pumping with 412-Ft. Single-Stage Centrifugals

The City of Lynchburg, Va., has recently completed the installation of two emergency pumping plants that will keep the city supplied with chlorinated river water in case its mountain-reservoir supply is interrupted. In one of these plants there are two 3,000,000-gal. single-stage centrifugal pumps working against 412 ft. total head and showing on official test 79.5% efficiency.

Lynchburg is spread over the sides and tops of hills alongside the James River. On the highest hill in town is a reservoir having a capacity of 4,000,000 gal. and with its flow line 320 ft. above the river. In the center of this, the College Hill reservoir, is located a standpipe (Fig. 1) with its top 40 ft. above the reservoir level. About 100 ft. below the College Hill reservoir—that is, 220 ft. above the James River—is the Clay St. reservoir of 3,500,000 gal. capacity. About 22 mi. from Lynchburg is the city's mountain reservoir of 500,000,000 gal. capacity and with its flow line 143 ft. above the top of the standpipe of the College Hill reservoir. A 30-in. pipe line connects the mountain reservoir and the College Hill reservoir; this comprises about 16,000 ft. of steel pipe, 6,000 ft. of cast-iron pipe and the remainder of wood-stave pipe. The maximum pressure on iron or steel is 200 lb. per sq.in.; and on the wood, 100 lb. This line crosses under the James River once. It was completed in 1907 at a total cost, including the reservoir and dam, of $700,000. The line has a capacity of 8,000,000 gal. per day, while the present draft is 4,000,000 to 5,000,000. In some seasons the rate drops to as low as 3,500,000 per day. The city has a population of about 30,000.

The pipe line from the mountain reservoir goes to the center of the College Hill reservoir and up the standpipe to the top. The flow passes downward between this supply and the standpipe shell, flowing into the reservoir over the stonework, as shown in Fig. 1. This arrangement permits supplying a portion of the city known as Rivermont—the best residential area and about one-third of the city.

The College Hill reservoir supplies directly about one-third the town. Part of its water flows down through a pipe to the center of the Clay St. reservoir, 100 ft. below, this last in turn feeding the lowest third of the city at a permissible pressure. It is seen that with this arrangement, if the pipe line breaks, the Rivermont district is at once out of water—although the other two-thirds of the city would be supplied for something more than one day.

FIG. 1. COLLEGE HILL RESERVOIR AND RIVERMONT STANDPIPE, LYNCHBURG, VA.

To insure a continuous supply for the highest district, the station and equipment illustrated in Fig. 2 were installed. This station draws from the College Hill reservoir and returns to the top of the standpipe through the terminal of the 30-in. main, the mountain-supply valve being closed during such operations. The College Hill pumping station has in it one 8-in. single-stage double-suction bronze-fitted centrifugal pump, mounted as shown in Fig. 3,

midway between a 50-hp. squirrel-cage 900-r.p.m. three-phase motor and a four-cylinder 5½x6-in. gasoline engine, completely equipped for operation at any moment. The engine, pump and motor are connected by Francke pin couplings, the pins being left out ordinarily between the engine and pump, as the motor is commonly used to drive. In case the electric power is off, the engine-coupling pins are slipped in and the pump and motor rotors are revolved together. This pump is designed to deliver 3,000,000 gal. per day (2,100 gal. per min.) against 60 ft. total head at 865 r.p.m. On official test the pump efficiency was shown to be 79%.

To take the place of the mountain supply in case of an interruption of more than a few hours, resort is had to the old scheme of pumping from the James River. Years ago, before the installation of the gravity system, all the supply was so pumped—by water-power reciprocating units. The same flume that supplied the water for hydraulic power has been converted into a suction bay for a new motor-driven station. The old cast-iron 18-in. pipe line, 5,200 ft. long and running from the river to the center of the College Hill reservoir, has been used for the emergency service.

In the station at the river two 3,000,000-gal. motor-driven units have been placed, taking current from the local electric station. Each motor has its own control panel with circuit breaker, inverse-time-limit relays and indicating instruments. A watt-hour meter records all the input to the plant.

The static head of operation is 354 ft. On account of friction the pumps were designed to deliver 21,000 gal. per min. against 412 ft. total head at 1,765 r.p.m. Since one unit would have to operate mostly alone, the point of maximum efficiency was shifted later to 2,500 gal. per min. The official test efficiency alone was found to be 79.5%, compared with a guarantee of 78%. When operating in parallel, the pumps handled 2,200 gal. per min. at 78% efficiency. These two pumps are each 10-in. horizontal single-stage double-suction horizontally split centrifugals. Each is connected to a 300-hp. 1,800-r.p.m. slip-ring motor. The cost of this equipment was less than that of three-stage volute pumps or two-stage turbine pumps and is thought to be the highest-head single-stage equipment in such service in this country. A liquid chlorine disinfecting equipment has been installed in this station for treating the river water.

These plants were designed by H. L. Shaner, City Engineer, and the construction was under the supervision of R. A. Thomas, Assistant City Engineer. The operation

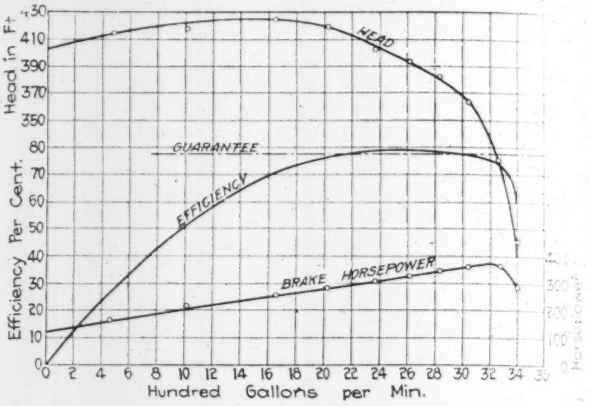


FIG. 5. TEST RESULTS OF LYNCHBURG SINGLE-STAGE 412-FT. PUMP

of the equipment is under W. C. N. Randolph, Jr., Superintendent of the Water Department. Wiley & Wilson, Consulting Engineers, of Lynchburg, were retained by the city as technical advisers. The pumps were built and installed by the American Well Works, of Aurora, Ill. The electrical apparatus and construction were by the General Electric Co., of Schenectady.

Automobiles, Motor Trucks and City Planning*

By Nelson P. Lewis†

In developing a city plan to meet not only existing conditions, but those which may arise in the future, it must be borne in mind that there is no one thing that will more powerfully affect such a plan than the change of conditions brought about by improved methods of getting from one part of the city to another.

In the case of vehicles devoted to pleasure traffic the higher speed and the enormous increase in their number involve questions of safety in the public streets and of traffic control. When most of our city streets were badly paved and only a few of the chief thoroughfares had street surfaces over which one could ride in comfort, there was serious congestion on these few streets. As our city streets become better paved and more attractive and as

*From a paper read before the City Planning Conference, Cleveland, Ohio, June, 1916. The paper in full appeared in the June issue of "The City Plan," the official organ of the National Conference on City Planning (19 Congress St., Boston, Mass.).

†Chief Engineer of the Board of Estimate and Apportionment, New York City.

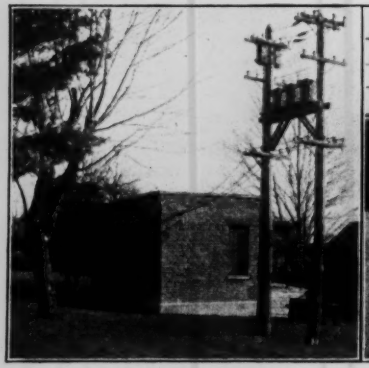


FIG. 2. COLLEGE HILL STATION

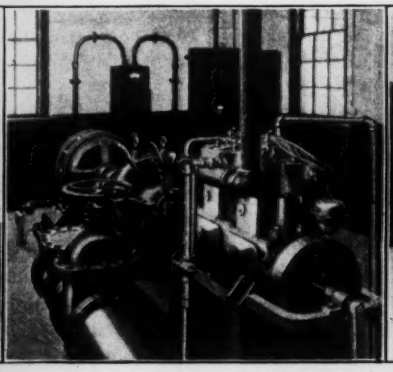


FIG. 3. COLLEGE HILL PUMP

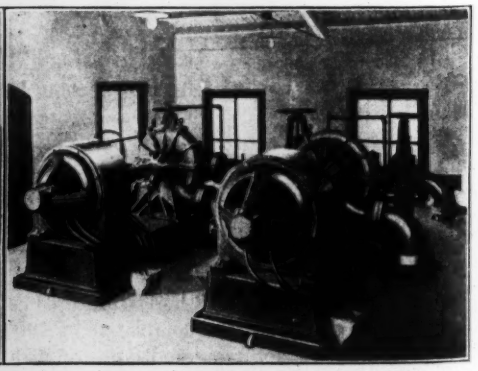


FIG. 4. SINGLE-STAGE 412-FT. PUMPS

the passage from one street to another is made easy by good connections and the avoidance of too abrupt turns and offsets, the motor car will be more and more disposed to follow the lines of least resistance and streets which are more attractive, even if it involves a greater distance.

So far as the obstruction of traffic is concerned, the small commercial vehicles present the same problems as do those designed for pleasure traffic, but in the case of the truck of large capacity an entirely different set of problems is presented, these being: Damage to pavements by reason of excessive wheel loads; obstruction to traffic by reason of low speeds; occupation of an undue proportion of the street, owing to their constantly increasing dimensions. Of the 221 manufacturers in this country producing commercial vehicles at the beginning of the present year 133 confine themselves to those of less than 3 tons capacity and none go above 7½ tons. These will not be considered in the following analysis. Of the 88 makers now offering trucks of more than 3 tons capacity, 58 have made no change in maximum capacity in their models for 1916; 12 have increased, 6 have decreased, and of 12 newcomers only two offer 5-ton and one offers a 6-ton truck. The popularity of the 3½-ton and the 5-ton trucks is quite noticeable.

Six of the largest makers of motor trucks, one organization dealing with the technical problems of their design and one devoting itself to the promotion of their use, have expressed their opinions as to the probable economic limits of weight of car, load capacity and load on rear axle. The maximum weight of car is estimated at from 10,000 to 12,000 lb., the load capacity at from 10,000 to 16,000 lb., and the load carried on the rear axle at from 16,750 to 24,000 lb. There is a very rational basis for the prediction that the maximum limit of weight will be about 6 tons.

Perhaps the most important consideration from the city-planning point of view is the size of the vehicle, more particularly its width and its flexibility. The manufacturers and others whose opinions have already been given with respect to loads and speed have indicated that the wheel base varies from 12 ft. 6 in. to 17 ft. 4 in., the over-all length from 22 ft. to 28 ft., the gage between centers of tires from 5 ft. 10 in. to 6 ft. 3 in., and the over-all width from 7 ft. 4 in. to 8 ft., the opinion also being expressed that these will be the probable limiting dimensions, although the width of special loads, such as plate glass carried flat, may exceed the over-all width in a few instances.

If a roadway in which there are no railroad tracks is designed to accommodate four lines of traffic, allowing 8 ft. for each line, which would permit 2 ft. of free space between the great majority of vehicles, and if trucks measuring 8 ft. over all are permitted on public streets, this width of 8 ft. for a line of moving vehicles will be insufficient. The roadway capacity will be actually reduced to three lines of traffic, and that means that the street capacity for accommodating vehicles will be reduced almost one-half by reason of the fact that the third or middle strip must accommodate vehicles moving in both directions, and the interference will be so great that the capacity of this middle strip will be very small.

The greatest traffic congestion commonly occurs on streets where there are costly buildings and where widening is out of the question. The obvious method of relief in such cases, where part of the sidewalk cannot be converted into roadway, is to establish detours or by-passes around such congested streets or districts, making such routes attractive by means of better pavements and easier grades, and even widening existing and cutting through new streets where the damage to property would be least.

The conclusions of the writer will be briefly presented, as follows:

1. The effect of the motor vehicle upon the city plan of the future, and that means upon city planning, will be beneficial in that it will aid in bringing about some of the most important results that city planners are striving to attain, such as:

(a) The discouragement of further centralization of population; (b) land values will be more diffused; (c) a better class of street and road improvements will be insisted upon.

2. While the amount of traffic on important thoroughfares will be greatly increased, the greater speed and flexibility of the motor vehicle will permit a much greater volume of traffic to be accommodated with safety in the same street, provided:

(a) That the speed and movement of all vehicles are controlled by intelligent police regulations; (b) that the width of commercial vehicles is so regulated that the number of lines of traffic which a street can accommodate will not be reduced and the capacity of the roadway thereby seriously curtailed; (c) that the wheel loads are so restricted that the sustaining power of street pavement and foundation is not exceeded. The most rational way to do this would be to prohibit wheel loads in excess of 700 lb. per inch-width of tire, and these only on rubber tires, the maximum loads on steel tires to be less, or say 600 lb. per inch of width.

3. The greatest congestion, delay and danger will be found where the traffic is most mixed. The traffic problem will, therefore, be simplified if heavy manufacturing, business and residences are confined so far as possible to certain well-defined districts.

4. Horse-drawn traffic requires relatively light grades. Motor cars and trucks can overcome much steeper grades and considerable expense can, therefore, be saved in regulating streets as they can conform more closely to the topography. The greater speed of the motor vehicle will also justify longer detours to avoid excessive grades, and little time will be lost thereby.

5. While street corners and junctions should be improved wherever possible, manufacturers of motor vehicles should understand that they must adapt their vehicles to street conditions wherever possible. The mobility of such vehicles, the short turning radius for the heaviest motor trucks, and the ingenuity which has been displayed in the design of trailers which will follow the tractor without the cutting of corners give promise that such adaptation to existing conditions is possible.

6. With decreasing cost of light motor cars and with the likelihood of cheaper fuel the number of such cars will doubtless continue to increase. The problem of their accommodation in public streets will solve itself with the improvement of the character of road surfaces, although a serious problem will be presented in providing for the accommodation of standing cars. This will undoubtedly require the widening of roadways where possible at the expense of the sidewalks and the provision at frequent intervals of parking spaces where cars may be left and cared for at a moderate charge.

Grit Chamber and Pump Station, Albany Sewage-Works--I

By John H. Gregory*

SYNOPSIS—Sewage-works to treat an average daily flow of 30,000,000 gal. and a maximum flow of 45,000,000 gal. consist of a gate chamber, bar screens, grit chamber, pumping station and Imhoff tank. Grit chamber is given cross-section and provided with proportional weir at outlet end, so as to give an approximately constant velocity. Clamshell bucket and dump-cars remove grit. The unique pumping station will be described in a later issue.

In a previous article[1] a brief description was given of the sewage-disposal works now under construction by the City of Albany, N. Y., to prevent a continuation of the offensive conditions in the Albany Basin, a small land-locked harbor that the city has improved at considerable expense, and also to prevent the excessive pollution of the Hudson River in general along the Albany water-front. It is the purpose of the present article to describe the Westerlo grit chamber and sewage-pumping station which are being built in connection with, and immediately adjacent to, the disposal works and which contain a number of features that may be of especial interest to engineers having to do with the design of such structures. A very much smaller station, known as the Patroons pumping station, is also being built for pumping the sewage from North Albany, a district which is not tributary to the intercepting sewer by gravity.

It may be well to state here that the disposal works provide, for the present, only for the clarification of the sewage by Imhoff tanks, but are so arranged that additional treatment works can be added at any time in the future without rearrangement or reconstruction of the present installation. The disposal works now under construction have a capacity to treat an average daily sewage flow of 30,000,000 gal., with an assumed maximum rate of flow of 45,000,000 gal. per 24 hr. With the works enlarged to the ultimate extent for which they are laid out, which is 50% in excess of the present capacity, they would then be able to treat an average daily sewage flow of 45,000,000 gal. and with an assumed maximum rate of flow of 67,500,000 gal. per 24 hr.

Albany is sewered on the combined system, and it is planned to pump and treat only the dry-weather sewage flow. It is possible, however, during the early years of operation to handle some storm water when the maximum rate of dry-weather flow is less than that for which the works are designed, and it will always be possible to handle some storm water runoff, provided it does not occur at the same time of day as the maximum rate of dry-weather flow.

At the disposal works the sewage first passes through a gate chamber, thence to and through a grit chamber, first being screened at the inlet end of the grit chamber.

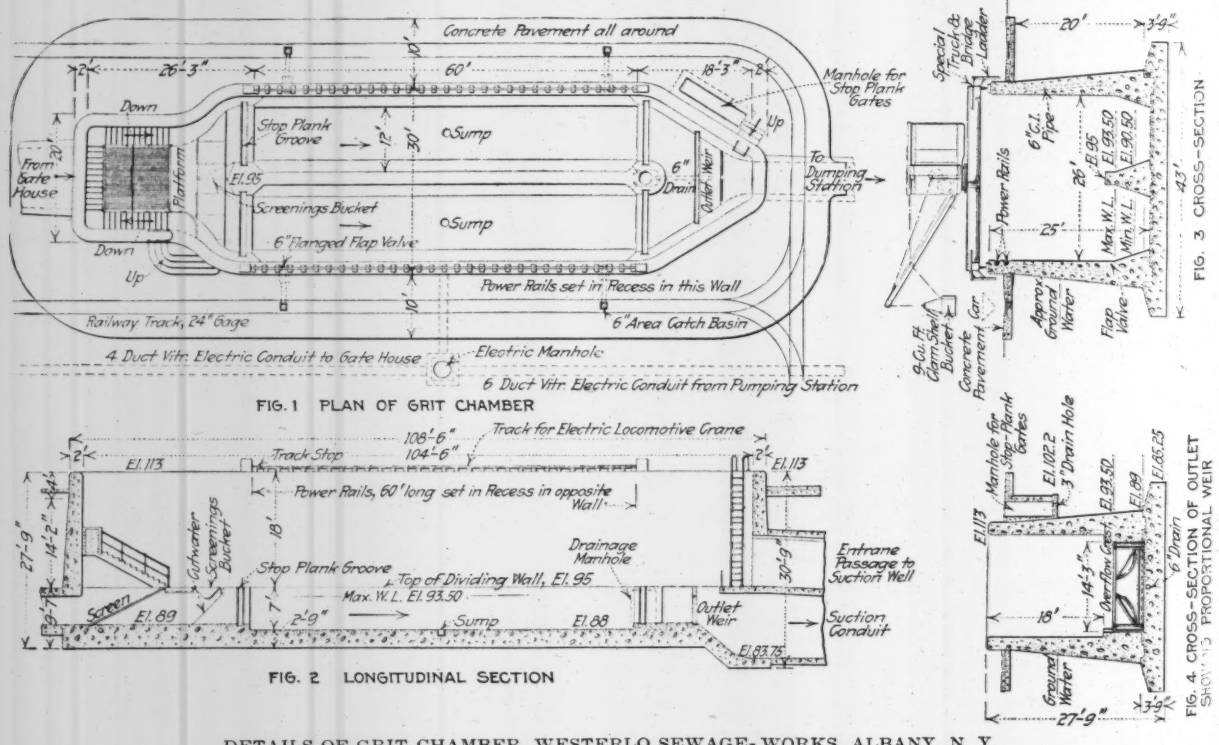


FIG. 1 PLAN OF GRIT CHAMBER

FIG. 3 CROSS-SECTION

FIG. 2 LONGITUDINAL SECTION

FIG. 4. CROSS-SECTION OF OUTLET SHOWING PROPORTIONAL WEIR

DETAILS OF GRIT CHAMBER, WESTERLO SEWAGE-WORKS, ALBANY, N. Y.

*Consulting Engineer and Sanitary Expert, 170 Broadway, New York City.

[1] "Engineering News," Oct. 7, 1915, p. 692.

and then through a suction conduit to the sewage-pumping station, from which it is pumped to the Imhoff tanks.

At the site of the works the ground is below maximum flood level in the river, and a levee has been built for safety. A levee is also being built to protect the gate chamber, grit chamber and pumping station, and in addition the walls of both the gate chamber and the pumping station are carried up above flood level. The walls of the grit chamber, however, are below maximum flood level, as the level of the sewage in the grit chamber during high water can be controlled at the gate chamber.

The gate chamber is a small structure built at the terminus of the intercepting sewer; and it is possible here to throttle the sewage flow or shut it off entirely by means of an electrically operated sluice gate controlled either from the gatehouse above or from the pumping station, as may be desired. With the flow throttled, or shut off entirely, the storm-water overflows on the lateral combined sewers, from which the intercepting sewer receives the dry-weather flow, will automatically come into action and afford an outlet to the river.

Screens and Disposal of Screenings

The plan and longitudinal section of the grit chamber are shown in Figs. 1 and 2, and cross-sections in Figs. 3 and 4. A coarse screen is placed at the inlet end, through which the sewage passes before entering the grit chamber proper. This screen, which is set at an angle of 27° with the horizontal, consists of ½x3-in. steel bars spaced 2 in. apart in the clear and is made up in sections that are removable so that the spacing of the bars can be changed should operating conditions render such a change advisable.

As the screen is planned to remove only the coarsest of the floating and suspended matters and only such as might injure the pumping machinery and as it will not require constant attention, it will be cleaned by hand, the operator standing on a platform against which the upper end of the screen rests. Access to the platform is from above by a stairway starting at the top of the grit-chamber walls.

The screenings, after being raked up onto the platform, are pushed into a so-called screenings bucket which is set in an opening in, and whose top is flush with the top of, the platform. The bucket has a capacity of 18 cu.ft. and consists of the body of a dump-car of the same size and type as those provided for transporting grit away from the grit chamber and sludge from the sludge-drying beds. The bottom of the bucket and also the sides for a height of 12 in. up from the bottom are perforated so as to allow the screenings to drain. The bucket, which is properly fitted therefor, will be raised by a locomotive crane and set on the running gear of, or its contents tipped into, a dump-car. The screenings will then be transported to the point of final disposal.

The grit chamber proper, in which it is intended to remove only the coarser and heavier mineral matters, consists of two compartments, each 12 ft. in width at the maximum high-water line and 60 ft. in length. The compartments are separated by a low dividing wall and have the same cross-section; under ordinary operating conditions both will be continuously in service.

The cross-section of the compartments, assuming the bottom of the cross-section to be at the same elevation as the invert of the inlet sewer, was so designed that, with the inlet sewer running about one-half full, corresponding to the average daily rate of dry-weather sewage flow when the works have reached their ultimate capacity, the mean velocity through the chamber, with both compartments in service, will be about 1 ft. per sec. As the compartments of the grit chamber have a length of 60 ft., the period of retention in the chamber proper will be about 1 min.

Cross-Sections to Maintain Uniform Velocity

While it is possible to give a grit chamber such a cross-section that a mean velocity through it of 1 ft. per sec. can be maintained with variations in the rate of sewage flow above and below the average rate of flow, it will very often be found that the cross-section will be of such a shape that a structure having such a cross-section would hardly be built. From a practical standpoint, however, it is possible to give a grit chamber such a cross-section that the variation in velocity from a mean velocity of 1 ft. per sec. will not be so great as to be a serious matter unless the ratio of the present average minimum rate of flow to the future maximum rate of flow is extreme.

In the grit chamber at Albany, with an ultimate average minimum rate of flow of two-thirds and an ultimate average maximum rate of flow of one and one-half the ultimate average rate of flow, the velocity will vary from a minimum of about 0.90 to a maximum of about 1.14 ft. per sec. During the earlier years, when the rates of flow are somewhat less than the ultimate rates, the velocities through the chamber will be correspondingly less. For example, with rates of flow two-thirds the ultimate rates the velocities through the grit chamber will vary from a minimum of about 0.76 to a maximum of about 1 ft. per sec.; with rates of flow as low as one-half the ultimate rates the velocities will vary from a minimum of about 0.67 to a maximum of about 0.90 ft. per sec.

Proportional Weir and Automatic Pump Control

In designing the cross-section of, and in computing the velocities through, the grit chamber at various rates of flow it has been assumed that the rise and fall of the water surface in the chamber will be synchronous with the rise and fall of the water surface in the inlet sewer. To secure this hydraulic condition, a proportional weir is inserted at the outlet end of the grit chamber, the crest of the weir being set a certain distance above the invert of the inlet sewer and the sides of the weir being given such a shape that, with the water surface in the grit chamber at any given elevation above the crest of the weir, which elevation will practically be the same as that of the water surface in the inlet sewer, the weir will then discharge at substantially the same rate at which sewage flows from the inlet sewer into the grit chamber.

One other condition is required—that is, the level of the sewage in the suction well shall not rise above the crest of the weir. This second condition is secured under ordinary operating conditions by automatic control of the pumping machinery so that the rate of pumping corresponds to the rate of inflow of sewage into the well.

Proportional weirs are not new and have been used for several purposes, and rectangular weirs have been used

at the outlet end of grit chambers. As far as the writer is aware, however, the Albany grit chamber is the first one actually under construction in which a proportional weir is to be used for controlling the flow through the chamber so that changing hydraulic conditions below the grit chamber will not affect the velocity of flow through the chamber and thereby possibly defeat the purpose for which the chamber was built, although the writer designed a grit chamber for the City of Trenton, N. J., some two years ago, nearly as large as, and quite similar to, the Albany one and in which a proportional weir was included.

The grit pits are shallow, the bottoms of the pits being only 1 ft. below the invert of the inlet sewer. The shallower the grit pits the less they will affect the velocity of flow through the grit chamber, and as a general proposition shallow grit pits with frequent cleaning are to be preferred to deeper pits with less frequent cleaning.

Clamshell Bucket and Dump-Cars to Remove Grit

The grit deposited in the pits will be removed by a clamshell bucket of 9 cu.ft. capacity, controlled from above by an electrically operated locomotive crane having a special bridge and end trucks. The crane tracks or runways are carried on the outer walls of the grit chamber and have sufficient length so that all parts of both pits can be easily reached by the bucket. The electric current for operating the crane will be carried by two power rails set in recesses in, and near the top of, one of the outer walls of the grit chamber. By placing the power rails in recesses they are protected from any damage that might occur to them due to the raising and lowering of the bucket.

The bottoms of the grit pits are flat; to protect them from serious abrasion by the bucket, steel T-rails are embedded in the concrete, the rails being set with the flange uppermost and flush with the surface of the concrete.

Under ordinary conditions it is intended to keep both compartments of the grit chamber in service while grit is being removed; but should it be found desirable to shut off the flow temporarily through one compartment while grit is being removed from it, this can be done by inserting a wooden stop-plank gate in grooves either at the upstream or the downstream end of the compartment, the stop-plank gate being handled by the locomotive crane.

Should it be desired to unwater a compartment, this can be done by inserting stop-plank gates at both ends of the compartment and then draining the sewage out through a manhole located in the downstream end of the dividing wall. From this manhole a pipe leads to the suction conduit which in turn connects with the suction well in which the level of the sewage will be lowered by increasing the rate of pumping, to a point at which the draining of the closed-off grit-chamber compartment will take place.

The grit will be discharged from the clamshell bucket into dump-cars running on tracks located adjacent to the walls of the grit chamber and will then be transported either to the sludge-drying beds for resurfacing them after sludge has been removed or to the point of final disposal. The cars will be hauled and moved about by a small storage-battery locomotive that will serve also for moving the sludge dump-cars.

Calming Rough Water with Compressed Air

Breaking up waves by means of a "reef" of air is a method of beach and pier protection that has been experimented with for the past 12 years and has now proved its practicability, according to the inventor, Philip Brasher, of the Brasher Breakwater Co., 141 Broadway, New York City. Under the Brasher system compressed air is expelled from a submerged perforated pipe so placed as to protect the beach or pier. The air pressure *per se* is not a factor in the successful operation of the system; a sufficient volume of air is the prime requisite. Patent

FIG. 1. LOOKING LANDWARD FROM END OF EL SEGUNDO (CALIF.) PIER THE DAY BEFORE IT WAS WASHED OUT

for this method was granted in 1907, and since then a number of plants have been installed and the following questions investigated: How much air is required per foot of perforated pipe? How can this air be evenly distributed? Will the perforated pipe fill up with sand? How much will the system cost?

Seven plants have been placed under various conditions of mud, sand and rocky bottoms and depths of water ranging from 17 to 50 ft. The latest plant to be established was one in front of the Standard Oil Co. pier at El Segundo, Calif. This pier, originally 4,000 ft. long, was badly damaged in the winter of 1914-15, when nearly 2,000 ft. of it was washed away. In an endeavor to protect the remainder of the pier the Brasher system was tried. This consisted of three sections of pipe, as shown in the sketch (Fig. 4), the first section, 120 ft. long, being laid in front of the pier and at a distance from

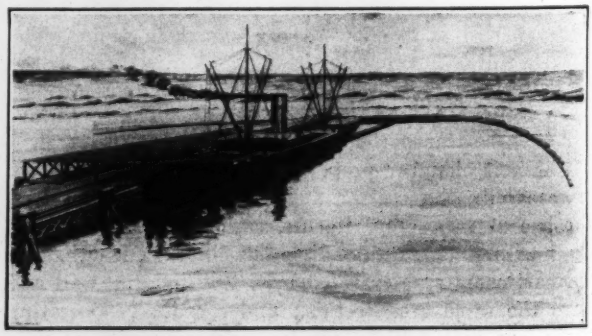


FIG. 2. ESTIMATED CONDITIONS AT END OF STANDARD OIL PIER (EL SEGUNDO) IF PROTECTED
With overlapping pipes on sand bottom

the pierhead of 145 ft. The other two sections are 100 ft. each and lie at right angles to the pier, one on each side, starting from the pierhead. The small cost of operation of this plant is shown by the fact that in 23 hr. operation during the height of a heavy winter storm the cost was only about $60. At the point where the pipes were laid the bottom was sandy and the water 30 ft. deep.

The particular installation described is of course inadequate to protect much more than the pierhead. A more elaborate design has been drawn up for this location. Fig. 2 is an idealized view of conditions that should obtain when it is in service. An extensive system of pipe is provided and is so controlled from the pier that the air can be blown into any particular section, depending upon the direction of the storm.

The several plants installed up to date cost from $10 to $20 per ft. protected. Navigation is not impeded, as the pipes lie on the bottom. It has been determined that with ¼-in. perforations the pipes do not fill with sand. The sand accumulates above them when not in use; but when the air is turned on, the sand is blown away. In protecting bathing beaches the invisibility of the system has another advantage. The distributing pipes usually are 4 in. in diameter. The ¼-in. holes are spaced 6 in. apart. From 3 to 10 cu.ft. of free air is required per lineal foot of beach protected.

At Brighton Beach, L. I., one of these systems was put in about six years ago partly for experimental purposes and partly to protect the bathing beach. The air compressor was operated by an induction motor using central-station current. The surf at this point is not very heavy, and after two years the plant was taken out. Pipes were laid at a depth of 17 ft., to protect 600 ft. of beach.

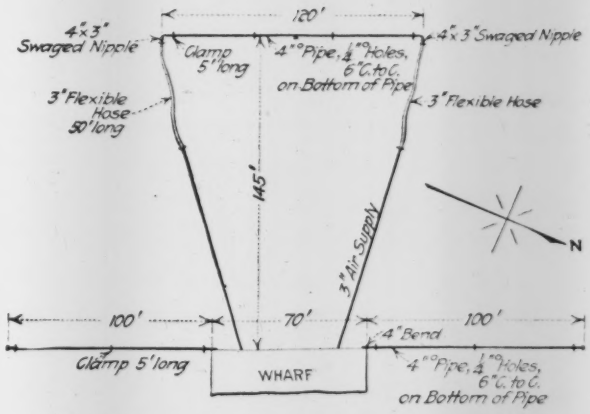


FIG. 4. EXISTING AIR-BREAKWATER EQUIPMENT AT EL SEGUNDO

On the Pacific Coast hundreds of miles separate the few safe harbors from one another. There are a number of piers at various locations, but the landing of vessels is a precarious operation and one that is not infrequently interrupted. If the Brasher system furnishes at reasonable cost safe harbors in time of storm, its commercial success is certain. Mr. Brasher sums up the advantages of his system as follows: "Thus protected, dredges can be kept working in exposed places during rough weather. The erection of seawalls, piers, lighthouses, etc., may be continued steadily. Partly completed structures may be protected until finished. Lightships can ride out the roughest gale in an artificial lagoon of calm sea. Stranded vessels can be protected from the pounding of the waves until floated."

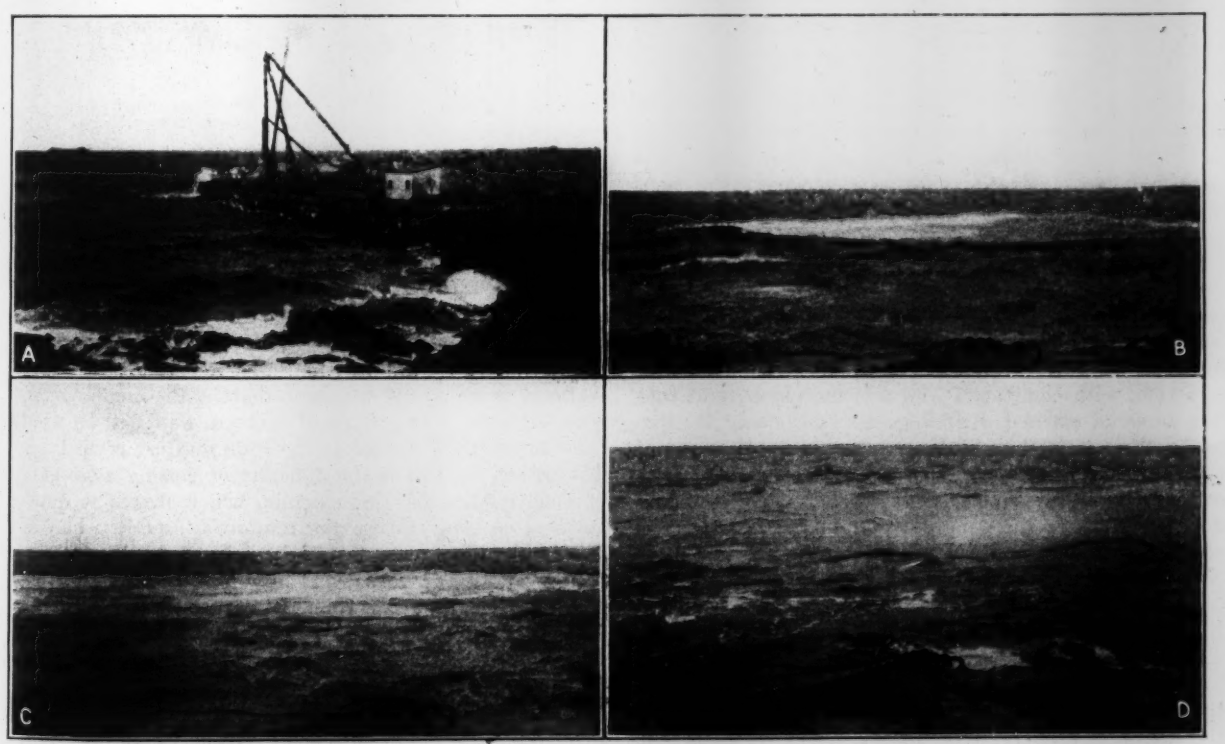


FIG. 3. SHOWING THE EFFECT OF THE BRASHER SYSTEM OF THE PIER AT GLOUCESTER (MASS.) QUARRY
A—Air off. B—Air just turned on. C—After few minutes of operation. D—Air just turned off

Snowsheds and Tunnels on the Great Northern Ry.

SYNOPSIS—About $1,500,000 to be expended on snowsheds this season, including both concrete and timber construction. Mostly double-track work. Also tunnels and concrete galleries for changes to improve location.

On the line of the Great Northern Ry. crossing the Cascade Mountains, in Washington, extensive improvements have been made to reduce the serious troubles caused by snow. These troubles include the blocking of the line by heavy snowfalls, and both blocking and damage by heavy snowslides. With increased traffic, very large expenditures have been warranted to protect the line and prevent interruption to train service.

The main part of the work consists of snowshed construction, with changes of line at some places, and tunnels to avoid exposed sidehill points and enable the line to be thrown farther back into the hill. In view of the heavy traffic all work on permanent location or relocation is built for double track, single-track work being done only where there is prospect of relocation. The snowsheds are of special interest in that for a considerable portion of the work the usual all-timber construction is replaced by a design having a concrete back wall and timber posts supporting the roof timbers. This is described below.

Where the railway loops down the western slope of the mountain for ten miles between Tye (at the end of the Cascade tunnel) and Scenic, about 58% of the distance is now protected against snow troubles. With the work to be done this season, about 76% will be thus protected. Nearly $1,500,000 will be spent on snowshed construction, while other improvements will bring the total cost of the season's operations to about $2,000,000. The work includes the following items:

On the east slope, 8,000 lin.ft. of new timber snowsheds between Leavenworth and the Cascade tunnel, $500,000. On the west slope, 3,700 ft. of combination concrete and timber snowsheds (3,000 ft. of which replaces old timber sheds), $500,000; 6,100 ft. of new timber sheds (100 ft. replacing old sheds), $450,000; changes of line at bridges Nos. 398 and 402, including 450 ft. of double-track concrete arch gallery and 2,650 ft. of tunnel, $550,000. Total cost, $2,000,000.

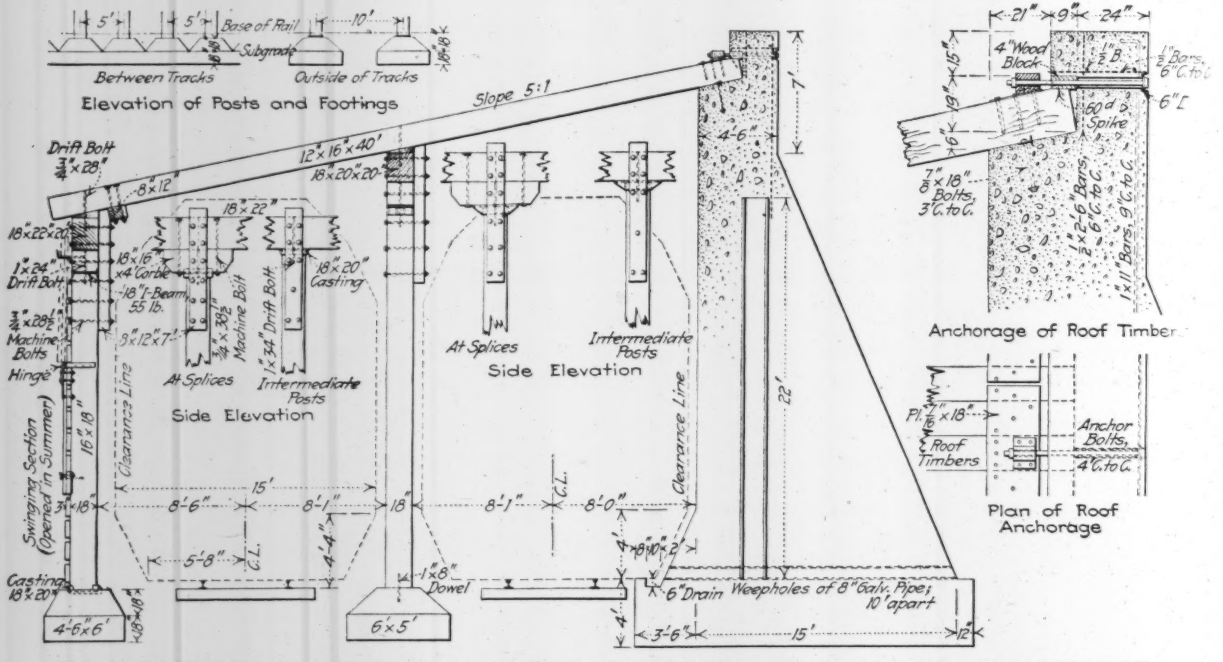


FIG. 1. SNOWSHEDS OF COMBINATION CONCRETE AND TIMBER CONSTRUCTION, GREAT NORTHERN RY.

The designs for the snowsheds are shown in Figs. 1 and 2. They are practically identical with those for the previous work on this division, described in *Engineering News* of June 4, 1914, except that in the combination snowsheds the gravity type of concrete wall will be used most extensively, instead of the reinforced-concrete type. The reason for this is the difficulty of getting aggregate of good quality for the concrete, which makes it questionable economy to use the reinforced section where a small amount of poor material might result in unsatisfactory results or perhaps failure to withstand a slide.

A point of difference from the earlier design is in the method of fastening the roof timbers to the concrete wall. The roof consists of 16x16-in. transverse timbers laid close together. In the earlier construction the ends of these timbers rested on and were drift-bolted to a 16x16-in. beveled timber set in a recess in the top of the wall. No trouble has developed with this form of fastening, but it has been thought best to strengthen this point

In the new design, shown in detail in Fig. 1, the roof timbers rest directly on the wall and are notched to fit over the projecting leg of a continuous 4x5-in. T-bar embedded in the concrete. The ends of the timbers are housed beneath a projecting ledge on the wall, with 4-in. wood blocks wedged between the timber and the ledge.

The roof timbers are securely anchored to the wall by horizontal 1½-in. bolts which pass through 2-in. sleeves and have their heads held in a 6-in. channel on the back of the wall. Upon the roof timbers is spiked a line of steel plates $\frac{7}{16}$x18 in., in lengths of 11 ft. 11 in., secured by drift bolts. Upon these plates are riveted socket castings for the anchor bolts, the nuts being screwed up against the castings. These anchor bolts are spaced 4 ft. c. to c.

The concrete wall will be built in sections not more than 48 ft. long, with no bond between adjacent sections. The shed as shown is designed for a load of 1,500 lb. per sq.ft. For a load of 1,000 lb., the roof timbers are 12x12-in., the outer and inner posts are 12x16-in. and 16x20-in. respectively, and all posts spaced 10 ft. c. to c.

The construction of the timber snowsheds is shown clearly in Fig. 2 and needs little description. The bents are of 12x12-in. timbers, with plank braces; they are spaced 4 ft. c. to c. where heavy slides are to be expected and 8 ft. c. to c. for lighter slides. Where the ground is flat on the outer side of the shed, the crib support for the outer posts is omitted and the roof is extended over an additional panel, as shown by the dotted lines.

In rough and steep ground, cribbing is used to support the intermediate posts on the uphill side. In flat earth slopes, however, these posts rest on caps and continuous sills, like the main posts of the bent.

Tunnels and Galleries

The improvement of the line through the Cascade Mountains this season will include changes of line at two points. Each of these will eliminate a bridge and some curvature. At one of these changes there will be two tunnels, 2,050 and 900 ft. long, with 250 ft. of concrete arch gallery. The other will have a 600-ft. tunnel and 200 ft. of arch gallery.

These galleries form a specially interesting feature of the work, being practically double-track tunnels built in opencut in sidehill ground and forming protected approaches to the rock tunnels. The approximate maximum section of gallery is shown in Fig. 3. The interior section is similar to that of the tunnel, noted below.

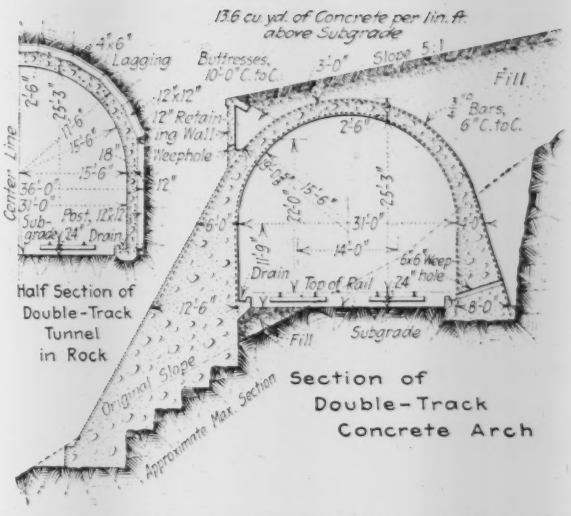


FIG. 3. CONCRETE GALLERIES AND DOUBLE-TRACK ROCK TUNNELS, GREAT NORTHERN RY.

The roof is reinforced with steel bars, and the side walls are carried down to broad footings.

To protect the gallery from slides, it is covered with backfilling, finished to a slope of 1 on 5 extending from the natural surface of the hillside. The fill is 3 ft. deep over the arch, and its toe is held by a 12-in. reinforced-concrete retaining wall, with buttresses 10 ft. apart. Weep holes in this wall provide for drainage of the fill.

The double-track tunnel in rock, Fig. 3, is of wide section, the clear width between walls being 31 ft., with tracks spaced 14 ft. c. to c. The roof is semicircular, giving a clearance of 25 ft. 3 in. above the rail level at the center line and 22 ft. above the outer rail of each track. The tunnel is timbered, with posts and roof segments 12x12 in., and 4x6-in. lagging over the roof. The bents are spaced 4 ft. c. to c. The concrete lining is 18 in. thick in the walls and 30 in. in the arch, these being the clear dimensions inside of the timbering. The walls are shaped to form concrete gutters or drains along the footings.

The top of the rail is 2 ft. above the subgrade. This tunnel section requires for each foot of length a neat excavation of 36½ cu.yd., and 7.13 cu.yd. of concrete for the lining. The single-track tunnel section in rock is 17 ft. wide and 24 ft. high, with a semicircular arch. Its area is 377 sq.ft., requiring 14 cu.yd. of excavation per lineal foot.

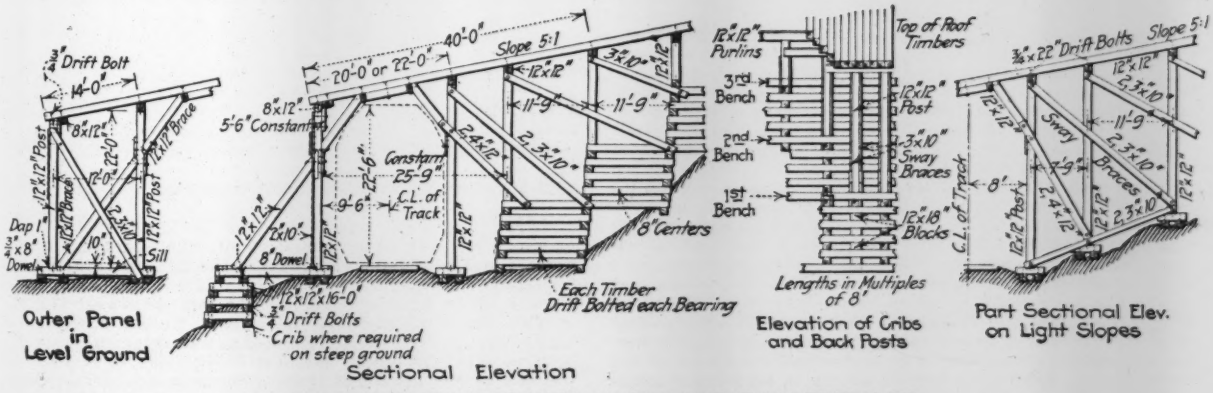


FIG. 2. TIMBER SNOWSHEDS ON THE GREAT NORTHERN RY.

Contracts have been awarded to Grant Smith & Co., of Spokane, Wash., W. J. Hoy & Co., of St. Paul, Minn., and Henry McPhee & Co., of Seattle, Wash. The work is under the direction of A. H. Hogeland, Chief Engineer, Great Northern Ry., St. Paul, Minn., and O. S. Bowen, Principal Assistant Engineer, Seattle, Wash.

Building a New Type of Dam in the Colorado River

In taking water from the Colorado River for the irrigation of the Imperial Valley, a problem of much difficulty has been the maintenance of the supply to the intake during the low-water season. A paper is to be read before the American Society of Civil Engineers on Sept. 26 (published in the "Proceedings" for May), on the "Control of the Colorado River and the Protection of the Imperial Valley." The author of the paper, J. C. Allison, of Calexico, Calif., is Chief Engineer of the California Development Co. Mr. Allison describes how in the low-water period of 1910 it became necessary to build a temporary rock weir across the Colorado River to raise the level high enough to maintain the flow in the intake. This weir cost $32,350 and raised the water a maximum of 3.4 ft. It was maintained during the season of low water with great difficulty and expense. In March, 1911, the trestle of the dam was removed, and the War Department forbade the dumping of further material in the river channel.

In 1915 a new type of hydraulic fill diversion dam was built across the channel of the Colorado. Mr. Allison describes the experience which led to the building of this dam and the methods adopted in its construction as follows:

Sole dependence for a sufficient water supply for Imperial Valley after 1910 was placed on the hydraulic dredge "Imperial," operating below the headgates, and on the dredge "El Centro," operating above the headgate in the intake canal and in the Colorado River channel, controlling sand bar obstructions, etc.

The actual computed yardage of solid material moved by the dredge "Imperial" from the canal below the headgate in one season's operations is equal to just one-tenth of 1% of the solid matter discharged through Hanlon headgate in the same interval, but it is also a fact that it was just this one-tenth of 1% of solid matter which had heretofore blocked the canal delivery below the headgate before the dredge was put in operation.

The major part of the one-tenth of 1% of solid matter removed by the "Imperial" was shingle, ranging from 1-in. rock to ¼-in. gravel, and sand ranging from gravel down to materials which are retained on a No. 10 mesh screen. It was the discovery of this fact that led to the successful operation of the Hanlon Heading, from the time of the entire removal of the rock weir in 1911 up to the present. This grade of material has been removed systematically every year since the dredge has been in operation, thus permitting the bed of the canal below the headgate to be maintained at an elevation which gave a sufficiently low water surface in the canal at all times. All other sands and silts not removed by the "Imperial" are carried down the canal in suspension.

The fact that there was deposited below the headgate, at times when the velocities in the intake exceeded 6 ft. per sec., material of a kind which could not be carried down the canal by the velocities below the headgate, which are never greater than 4 ft. per sec., led to the discovery of a means by which a hydraulic-fill dam could be placed across the Colorado River at a point directly below the intake, to serve as a means of diverting sufficient water into the Imperial Canal, should the necessity ever again arise. This time did arrive during the summer of 1915, because of the combination of an exceptionally low Colorado River during August and September, and an exceptionally great demand for water in Imperial Valley, due to the large increase in irrigated acreage over the previous years.

A study of the cross-section of the channel of the Colorado at Hanlon Heading, and the gaging station of the United States Reclamation Service at Yuma, shows that the Colorado—like other rivers forming deltas—scours and refills its channel bed with the increase and decrease of quantities of water, and that a direct relation exists between the character of the materials forming the bed and the velocities of the water. For example: The river bed is scoured to an elevation of 85 at Yuma, Ariz., by a discharge of 140,800 sec.-ft., whereas it is refilled to an elevation of about 110 by a discharge of 3,400 sec.-ft. The only reason that the bed does not scour deeper than an elevation of 85, with a discharge of 140,800 sec.-ft., is that at that depth the materials constituting the bottom are so heavy that the velocities, registered as 6.8 ft. per sec. for this discharge, will not move them. It follows that, in refilling the bed, the materials are graded as to weight, and deposited in direct relation to the velocities.

Though this is by no means a new discovery, the experience with the dredge "Imperial" below the headgate served to bring out more strikingly the real relations between the weights of materials and the velocities, and showed a way to make a practical use of this knowledge.

The scour line of the bed of the Colorado at Hanlon Heading (the intake of the canal) is at an elevation of about 84. The ladder of the 10-in. suction dredge "El Centro," operating in the river and above the concrete heading, was designed so as to make it possible to pump from an elevation of 80 when the surface of the river was 110. The materials in this area below the scour line had withstood the scouring effects of velocities as high as 7 and 8 ft. per sec. Those materials, when brought to the surface and deposited in the water where the velocity had decreased to 2 and 3 ft. per sec., due to falling discharge, were certain to stand in the position placed by the discharge of the dredge, making it possible to construct a hydraulic-fill dam under the water of a running stream varying in depth from 2 to 15 ft.

HYDRAULIC-FILL DIVERSION DAM

On Aug. 12, the "El Centro" commenced the construction of such a dam below the intake, and in 14 days had raised the fill to an elevation of 12 in. above the water surface three-fourths the distance across the river. This fill reached from the Arizona bank of the river to within 250 ft. of the California shore, or a total length of fill of 650 ft. Due to the contraction of the channel during this period, a head of somewhat more than 1 ft. was developed, which resulted in a corresponding increase in velocity through the remaining opening. The velocities resulting from this head became as great as those developed during flood discharge of the river, and consequently material pumped from below the scour line could no longer be deposited in the dam with any degree of economy, without adopting some method of decreasing the velocities. In closing the last 250 ft., therefore, a great quantity of brush and sacks filled with gravel was used.

The closure of the final 75 ft. of the channel represented the most difficult operation in the construction of the dam, and the means devised for making this closure again demonstrated the advantage of close observation of the action of a river when left to itself, and the application of the results of this study in a practical way.

From Aug. 12 to Sept. 12 was the period of principal construction. The time subsequent to Sept. 12 was spent principally in making the final closure and in maintaining the structure. On Aug. 12 the river was discharging 8,800 sec.-ft. Of this discharge, 2,529 sec.-ft. were flowing into the Imperial Canal, all the gates in the heading being fully opened. The requirement for irrigation in Imperial Valley was 5,000 sec.-ft. By Sept. 12, the river discharge had dropped to 4,000 sec.-ft. During this period of 30 days there was used in the construction of the dam:

40,000 cu.yd. of sand and gravel fill, at 2.7c. per yd.	$1,080.00
21,300 burlap and canvas sacks, at 3 to 6½c. each...	1,140.00
450 cords of willow brush and poles, at 73c. per cord	328.50
800 lb. of wire	32.00
Cables and clamps	100.00
Labor, filling and placing sacks, carrying and placing brush, poles, etc.	2,000.00
	$4,680.50
Plus 10% for superintendence and tools, etc.	468.05
Total	$5,148.55

Since Sept. 12 there has been an additional expense of $2,000 for raising and strengthening the dam by dredging, and for labor in closing and opening the dam, and general maintenance of the structure, making the total final cost $7,148.55.

The following methods were used in constructing the major part of the dam: Pumping was commenced on the Arizona side, the dredge pontoon being some distance above the site of the dam, and the discharge being carried astern through a 10-in. pipe floated on pontoons to the fill. Owing

to the admixture of lighter grades of material encountered in burying the suction pipe to the scour line depth, and to the subsequent caving in on the suction of the surface materials when the depth was once reached, the base of the dam, in a great many places, became as wide as 150 ft. The heaviest materials pumped—in some instances stones and cemented adobe chunks 6 in. in diameter—remained exactly where placed, but the lighter sands and gravel were carried by the current some distance down stream before they lodged in permanent position. When a point near the water surface was reached with the fill, the conservation of the material was made possible by constructing brush fences to help retain the material at the slope required. By this process the structure was carried across the stream to the point of final closure. Three re-runs across were made later with the dredge, bringing the crown to an elevation of 114, which is about 5 ft. above the water surface of the river when 5,000 sec.-ft. are passing into the canal.

On Sept. 10, 2,300 sec.-ft. were passing through the opening in the dam, and 3,300 sec.-ft. had been diverted into the Imperial Canal. The head on the dam was 2 ft., and the resulting velocity through the remaining opening was 8 ft. per sec. The bottom of the opening had been paved with about 10,000 sacks, filled with gravel pumped by the dredge, and stoutly tied, decreasing the depth to about 5 ft., and thereby preventing scour. Great quantities of sacks were used in protecting the Arizona end of the opening, the California shore, as before stated, being protected by its rock facing. Additional sacks were used in forming the overpour riffles as fast as the head developed. A great number of the sacks were of no avail, either because of their lodgment in the wrong place, or because of breaking and washing out; and, throughout the whole process, constant replacements were required because of the boring effects at the toe of the sack riffles, causing the embedding of the sacks.

In paving the bottom, the point above the opening at which the sacks should be dropped in order that they might lodge on the bottom in the proper place, was determined by constantly experimenting with sacks thrown overboard attached to wires, thus making it possible to locate the sack when it had finally settled by following the line of the wire. Only those filled with the coarsest shingle and gravel and of greatest weight would remain in position and withstand the velocity for any length of time.

In the final closure, when the bottom had been protected carefully, cables were stretched across, securely anchored to deadmen on the California side and in the body of the dam on the Arizona side. The first cables used were of ¾-in. plow steel, but proved to be too light when maximum strains were put on them. After one failure in making the closure, due to breaking the cable, a double strand of 1¼-in. cable was used for the lower run.

Willow poles were cut from the Arizona and California shores near-by, where the supply is abundant. The poles were 30 ft. long, the diameter of the large end being 10 in., and the small end tapering to 5 or 6 in. By the skillful use of barges and lines working above the openings, these poles were maneuvered with the current in such a manner as to cause their large ends to lodge against the cable stretched across the opening above the water surface, while the small ends rested on the bottom at an angle of 30°, much as a drifting snag will lodge in a swiftly running stream when the heavy end is dragging the bottom and the upper end encounters a surface obstruction of trees or other large drift. These poles were placed against the cable, in this manner, with enough space (sometimes 4 or 6 in. between them) to give sufficient strength to the structure and yet not decrease the area in the opening so greatly as to produce much additional head. When all the poles were in place, large bundles of willow brush were made up, weighted heavily with sacks, and rolled into the water from the barges, anchored some distance above the opening, in such manner as to cause them to strike the bottom of the channel and roll against the lower ends of the poles. As additional bundles were thrown in, the pressure against the poles increased, forcing them to bend, until in some instances, before final relief to the strain was obtained, the upper parts of the poles were standing vertically, and the lower sections were bowed to the extreme. Green poles were always chosen because of their flexibility.

The placing of the bundles of brush was carried on in sections only, and only in sufficient quantity to decrease the velocity through the section to a point where material deposited by the dredge could be retained in place. In this manner the strain against the poles in any section of the opening was of short duration, as the dredge would give relief in a few hours by extending the dam above. Of course, a great many sections failed during the work, because of poles breaking, but means were at hand for speedy replacement when such accidents occurred. Because the impounded area above the dam was so great as to require several hours to increase its elevation when flow through the opening was reduced, the extra heads created by stoppage with the brush were not felt on the poles to any great extent before the dredge had given relief.

After numerous local failures, due to small freshet rises in the river, and the breaking of cables and poles, the opening was entirely closed on Sept. 20, the head then being 6.4 ft. and the discharge of the river, measured at Hanlon Heading, 2,958 sec.-ft. Between Sept. 20 and 27 the river discharge fluctuated between 3,000 and 4,000 sec.-ft., all of it being required for irrigation in Imperial Valley. On Sept. 27, because of advance notice from Needles of a rise in the river, the dam was cut at a place provided, and all water, with the exception of 4,000 sec.-ft., was passed through into the channel of the Colorado River below.

The waters thus let through the dam were forced into the California bank below by local obstructions, so that on Oct. 2 the opening in the dam was again closed, and all the water was diverted into the Imperial Canal. The discharge of the Colorado River on Oct. 2, as measured at Hanlon Heading, was 4,406 sec.-ft. The second closing head was 8.2 ft. (The closing head of the Hind rock-filled dam, on Nov. 4, 1906, was 15.8 ft., with a river discharge of 9,275 sec.-ft.)

The 36 hour. following the second closure were spent in clearing obstructions in the channel immediately below the dam, and in providing a new waterway to train the flow away from the California shore. On the evening of Oct. 3, the dam was again cut at the place provided, because of a rise in the river in excess of the water requirements in Imperial Valley. During the period from Oct. 3 to the present writing, the opening in the dam, 100 ft. in width, has been left, and through it the discharge has been as great as 8,000 sec.-ft. In constructing the dam, provision was made for accommodating the freshet discharges of the river, which commonly occur during October and November. These sections will be sacrificed only as required, because of the fact that during December and January the discharge of the river commonly reaches the minimum, and this being a year of exceptionally low water, it is quite likely that the irrigation requirements of Imperial Valley will again make it necessary to close the dam fully. Entire removal probably will be advisable in March or April, 1916, and by that time it will have fulfilled its purpose as a means of diverting sufficient water to meet the irrigation demands of the Valley. In any event, the annual flow period of May and June, 1916, will remove the structure entirely. If the conditions which develop before this time should require it, the whole structure could be speedily removed by mining, as it is composed entirely of brush, sacks and gravel, with the exception of a few willow poles driven with the aid of a jet from the dredge. The structure contains absolutely nothing of a permanent nature.

The experience in building this dam has developed naturally a great many improvements which would be adopted in a second construction of similar nature; so that such a dam could be built in a much shorter period and with much less expense. For instance, by constructing an aërial tramway, anchored to towers on each bank of the river, dredge lines can be carried across the stream during its flood, thus enabling the dredge to venture into the river during much higher water, resulting in earlier completion of the work. The ladder of the dredge "El Centro" is to be lengthened 15 ft., enabling it to pump material at a greater depth below the scour line. These two improvements alone will permit the commencement of the dam while the river is discharging 56,000 sec.-ft., and will permit the complete diversion of a river carrying 15,000 sec.-ft. if necessary. More economical use of sacks and the labor attending can be achieved by a more general use of cable and pole manipulation in closing the final opening. Fewer accidents in the final closure will be assured by anticipating the material requirements in time to accumulate sufficient quantities of poles and brush. By lengthening the pontoon and pipe line of the dredge, thus providing an increased radius of operation, a large supply of material suitable for construction can be obtained.

Utilization of Material from Old Freight Cars is being practiced systematically by the Baltimore & Ohio R.R., and the mechanical department estimates a salvage of $20 per car in this way. The lumber is sawed and planed to remove defective portions and to give suitable sizes. It is then distributed for use in repair work and for building small structures, freight platforms, etc. Bolts, nuts, wheels, rods and other metal parts are sorted, trimmed and stored. The results have been so satisfactory that a reclamation plant has been established at Zanesville, Ohio, to which are sent all cars that are to be scrapped or destroyed. Reclamation work of a lighter character is carried on at division repair shops.

New York's Military Map of the Year 1778

By Frank B. Tucker*

As an example of the work of the military topographer of the closing quarter of the Eighteenth Century, the map herewith reproduced is an achievement to be proud of. The map itself is understood to be in the British Museum. A chance visitor to the Topographical Bureau, Borough of Queens (to whom due acknowledgement is here made), had a photographic copy of it which he kindly allowed to be photographed in turn. It is undoubtedly the first military map to be made of New York and its environs and possibly the only one; for *Engineering News*, no longer ago than Apr. 20 last, called attention editorially to the lack of such a map, citing only Charleston as a city with a military map, credit for which belongs to the Confederacy.

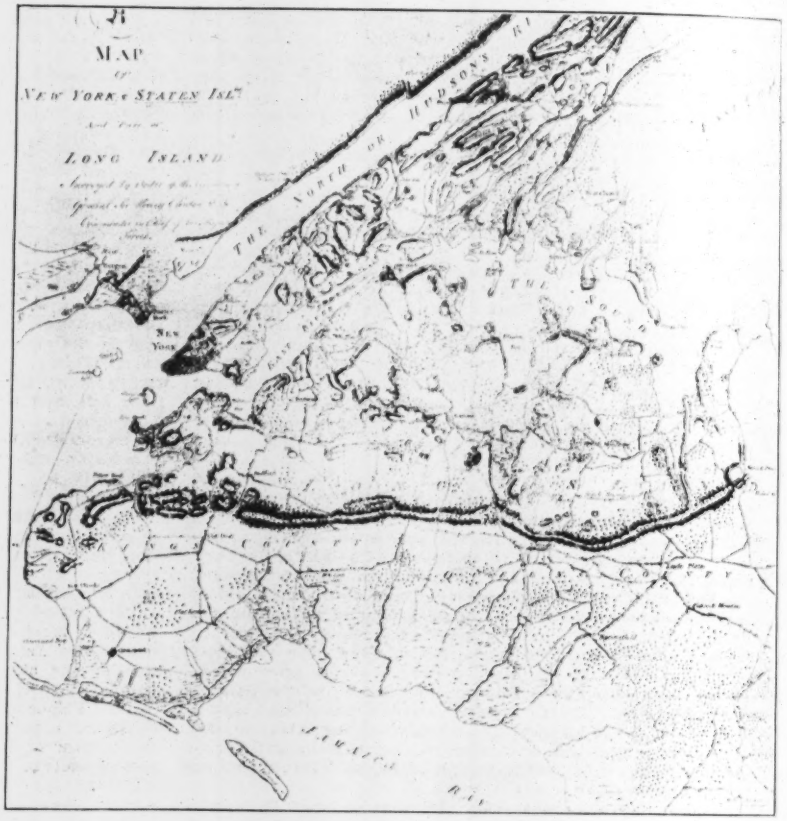
If ordered by Clinton as commander-in-chief, it was started sometime subsequent to May, 1778, the date on which he succeeded Lord Howe. Presumably, he sought to forestall being trapped in New York as Gage had been in Boston and as the Americans would have been in New York, had it not been for Washington's consummate skill in withdrawing therefrom.

While for military purposes the map has long ago outgrown its usefulness, as an example of the surveyor's art antedating present-day instruments of precision and as a source of historic information it well deserves careful study. The accuracy of the map is remarkable. Shore and coast lines, islands and hills, roads and trails are truly defined and readily identified on comparison with recent maps. The locality names that stud the map are in common use today and serve to distinguish different sections of the city. The highways that then obtained are for the most part the arterial thoroughfares of Greater New York. In some cases they have been widened and straightened to meet the needs of highly populated communities; in other cases they are still country roads with the same kinks and turns that Clinton's surveyors plotted.

Applying the scale of the map, it is evident that some system of control was used, distant points measuring remarkably close, nearer points only slightly less so. It may be that a number of fixtures were astronomically established and then connected up by traverses run with a compass and some kind of a perambulator.

To the student of American history the map is a key to many valuable data. Not alone does it show the topography and road system as it existed about New York in 1781, but it shows also the buildings that lined the roads, in a number of instances giving the occupant's name, as in the case of the Abthorpe mansion, where Washington was when the battle of Harlem Heights started. That the strategist of this period recognized the need of a thorough knowledge of the topography of the country as a basis for military preparedness, this map bears silent witness.

To the student of logistics the map is a true plan of the battles of Long Island and of Harlem Heights, which



BRITISH MAP FOR DEFENSE OF NEW YORK CITY, 1778

General Carrington in his "Battles of the American Revolution" sought to compile from records and descriptions of the battles. It was at Gravesend Bay, Aug. 22, 1776, that the British landed. On the twenty-seventh they moved northward along the several roads leading toward Brooklyn, to attack the Americans who guarded the passes of the roads through the ridge of hills, then heavily wooded, where now are Prospect Park and Greenwood Cemetery. The Americans succeeded fairly well in holding their own until Clinton and Cornwallis reached their rear by way of the Jamaica and Bedford road, foolishly left unguarded; then all was confusion. Washington, two days later, under cover of storm and darkness, crossed the East River from the fort on Brooklyn Heights (shown on the map) to New York. The British followed later from near Newtown Creek. The battle of Harlem Heights (about where Columbia College is now) was fought on Sept. 16. Washington then withdrew all except the garrison of Fort Washington, at the north end of New York island, to White Plains, licked the British at Chatterton Hill, then crossed to New Jersey. The British returned to New York, captured Fort Washington and continued in undisputed possession of the city until the end of the war.

*Topographical Bureau, Borough of Queens, New York City.

Wheel-Load and Impact Charts for Railway Bridges

By D. B. Steinman*

The accompanying three diagrams, constructed on the same plan as the writer's Chart of Equivalent Uniform Loads (*Engineering News*, Apr. 22, 1916), have been devised as additional time-savers in railway-bridge computations. The first of these is a diagram giving directly the position of Cooper's loading to produce the maximum value of any stress. The other two give the impact coefficients for any stress by the American Railway Engineering Association formula and the Lindenthal formula respectively.

All of these charts are based on the same principle, namely, that the critical wheel-position for any stress depends only on the lengths, l_1 and l_2, of the two segments of the influence triangle for that stress. Accordingly the three charts consist of curves plotted with l_1 and l_2 as coördinates, the values of l_1 and l_2 for different stresses, applicable to all the charts, being defined in the sketches on the first chart.

Chart of Critical Wheel Loads

This chart gives the critical wheel for any stress, i.e., the number of the wheel, in Cooper's loading, to be placed between l_1 and l_2 to make the stress a maximum. It will be found useful in engineering schools in checking the results obtained by the application of the various analytical and graphical criteria for load positions. It is superior to all such criteria inasmuch as these may be satisfied by a number of different wheels and there is no way, except exhaustive trial, to determine which wheel gives the greatest stress.

The principal use of this chart, however, is to serve as a basis for stress and impact computations and for the construction of other charts which depend upon load positions. Thus, as explained below, this chart was used as a foundation for the construction of the other two.

It should be noted that on or close to the boundary line between any two wheel-areas on this chart, either wheel may be selected, since the difference between the respective resulting stresses is so small as to require very fine computations to disclose it. At such points, therefore, the wheel giving the greater impact stress should be chosen.

This chart may be used for web stresses, as well as for bending moments, chord stresses, floorbeam reactions and pier reactions. In using it for web stresses, however, if wheel 12 or 13 should be indicated, it must be replaced by 3 or 4 respectively to avoid loading the negative portion of the influence diagram. The corresponding correction in Chart 2 is made by disregarding the area bounded by double lines.

The Impact Charts

The object of the impact charts is to save the time of finding the length of loaded train for maximum stress and of substituting this length in the respective impact formulas. Chart 2 is constructed for the American Railway Engineering Association impact formula, which is now the most widely used impact formula in this country. Chart 3 is constructed for G. Lindenthal's impact formula, which is fully explained in an article, "A New Impact Formula," in *Engineering News*, Aug. 1, 1912. The Lindenthal formula appears to be complicated; but by the use of this chart its application becomes very simple and expeditious.

The two impact diagrams were obtained, using the chart of wheel loads as a foundation, by drawing horizontal lines through each wheel-area to represent the successive impact coefficients and then replacing the resulting stepped graphs by continuous curves. This substitution of smooth curves for the broken lines, in addition to simplifying the two charts, serves to provide an automatic correction in those cases, which are missed in the ordinary application of the formula, where the governing wheel for maximum live-load stress should be replaced by another to produce the maximum total stress with impact included.

Either impact diagram may be drawn superimposed upon the writer's chart of equivalent uniform loads,[1] so as to give both the live-load and the impact stress at a single plotting of a point. If it were not for the usual preference to have these two stresses listed separately, a still greater saving of time could easily be effected by combining the A. R. E. A. impact and the equivalent load diagrams into a single set of curves giving directly the equivalent uniform load for total live-load stress with correction for impact already included.

Examples of Use of the Charts

The following examples are selected to show the time-saving value and other advantages of the three charts.[2]

Example 1—Required the critical wheel loads and impact coefficients for all the members of a double-track Pratt truss having six 25-ft. panels.

Referring to the three charts, the following figures are obtained:

Members	l_2	l_1	Critical Wheel (Chart 1)	Impact Coefficients (K) A. R. E. A. (Chart 2)	Lindenthal (Chart 3)
L0L2, L0U1	125	25	4 at L1	0.52	1.66
L2L3, U1U2	100	50	14 at L2	0.53	1.70
U2U3	75	75	12 at L3	0.52	1.68
U1L2	100	20	3 at L2	0.57	1.94
U2L3, U2L2	75	15	3 at L3	0.63	2.00
U1L1	25	25	4 at L1	0.77	2.00
L2U3, U3L3	50	10	3 at L2	0.70	2.00

Example 2—For a certain member in a double-track bridge, $l_1 = 40$ and $l_2 = 120$. Required the impact by the A. R. E. A. formula. For $l_1 = 40$ and $l_2 = 120$, Chart 2 gives the impact coefficient $K = 0.51$.

Calculation by means of the impact formula gives a smaller result, $K = 0.496$. This value is obtained by placing between l_1 and l_2, wheel No. 6, which is the critical wheel for maximum live-load stress.

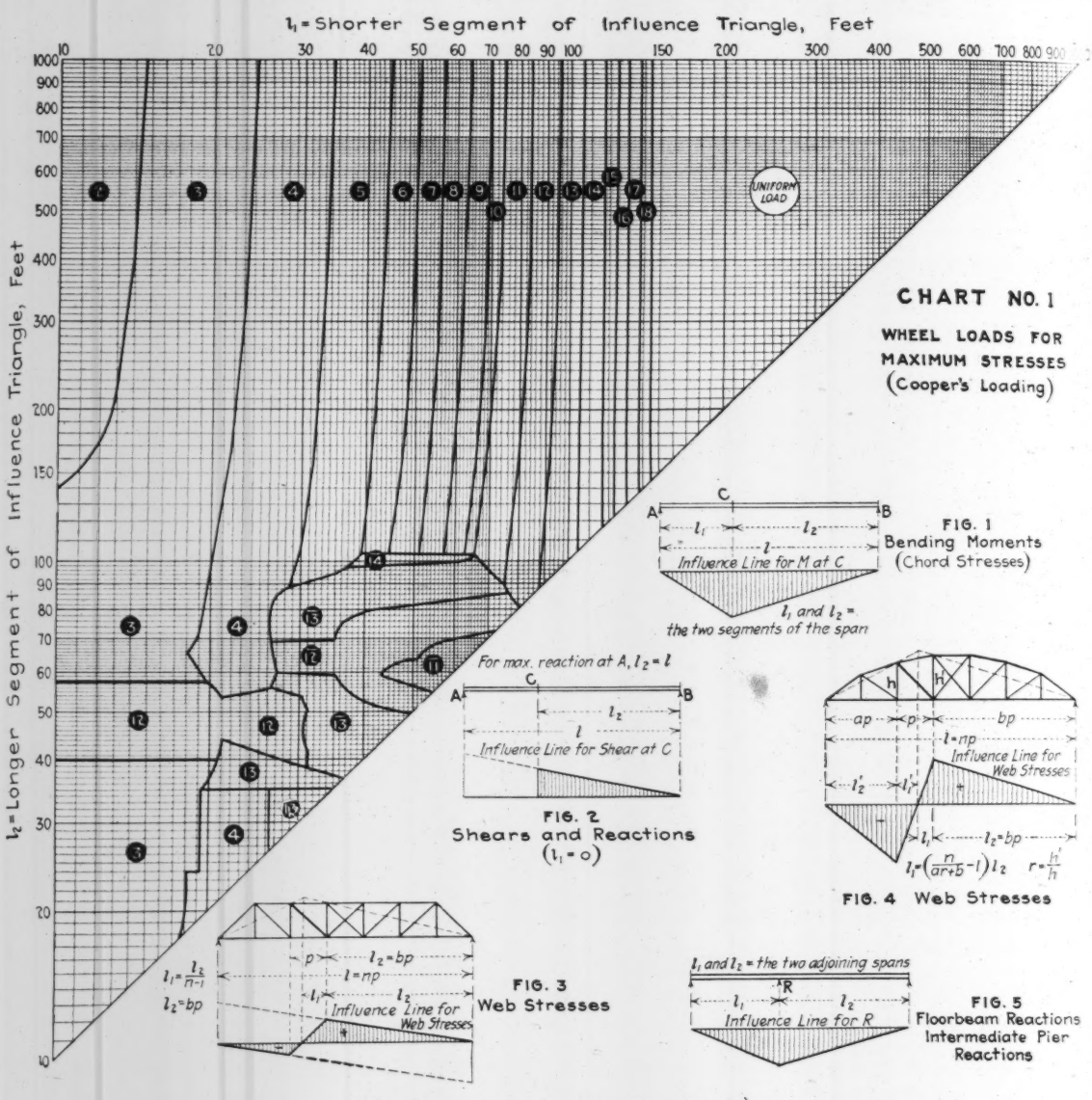
The reason for the above difference between chart and formula is disclosed by Chart 1; this shows that the point ($l_1 = 40$, $l_2 = 120$) lies near the boundary between the areas for wheel 5 and wheel 6, indicating that the two wheels produce nearly the same live-load stress. The impact chart automatically takes this fact into account and gives the impact coefficient which corresponds to wheel 5 ($K = 0.51$) since this will yield the maximum total stress. Similar results are obtained with Chart 3.

Example 3—Required the total bending moment for E60 loading at the center of a 120-ft. plate-girder span, weighing 5,200 lb. per lin.ft.

*Special Assistant Engineer with Gustav Lindenthal, Consulting Engineer, 68 William St., New York City.

[1] "Engineering News," Apr. 22, 1915.

[2] Large-size copies of any of the above charts, printed in black lines on linen, specially prepared for office use, complete with sketches and formulas, will be sent by the author to any address upon receipt of price, $1.


l_1 = Shorter Segment of Influence Triangle, Feet
l_2 = Longer Segment of Influence Triangle, Feet
10 20 30 40 50 60 70 80 90 100 150 200 300 400 500 600 700 800 900
10 20 30 40 50 60 70 80 90 100 150 200 300 400 500 600 700 800 900 1000
UNIFORM LOAD
CHART NO. 1
WHEEL LOADS FOR MAXIMUM STRESSES
(Cooper's Loading)
FIG. 1
Bending Moments
(Chord Stresses)
Influence Line for M at C
l_1 and l_2 = the two segments of the span
For max. reaction at A, $l_2 = l$
Influence Line for Shear at C
FIG. 2
Shears and Reactions
($l_1 = o$)
FIG. 4 Web Stresses
Influence Line for Web Stresses
$l_1 = \frac{l_2}{n-1}$
$l_2 = bp$
FIG. 3
Web Stresses
l_1 and l_2 = the two adjoining spans
Influence Line for R
FIG. 5
Floorbeam Reactions
Intermediate Pier Reactions

(These sketches apply to all three charts)

DIRECTIONS FOR USE OF CHART NO. 1

This chart can be used for bending moments, chord stresses, web stresses, floorbeam reactions and intermediate pier reactions.

This chart gives the number of the wheel to be placed between the two segments, l_1 and l_2, to produce the maximum stress

The shorter segment is ahead followed by the longer one except when the wheel number is overlined.

For $l_1 > 142'$, place head of uniform train load at 142' from head of span

DIRECTIONS FOR USE OF CHART NO. 2

Use l_1 and l_2 as defined in the above key as coordinates and interpolate between the curves to find "K". Take the value of "K" from the marginal column corresponding to the number of tracks on the structure.

Multiply the live load stress by "K" to obtain the impact stress.

For points on the double lines, use the higher of the two values of "K"

DIRECTIONS FOR USE OF CHART NO. 3

Use l_1 and l_2 as defined in the above key as coordinates and interpolate between the curves to find "K". Take the value of "K" from the marginal column corresponding to the number of tracks on the structure.

$\frac{L^2}{L+D}$ Multiply the value of "K" by to obtain the impact stress.

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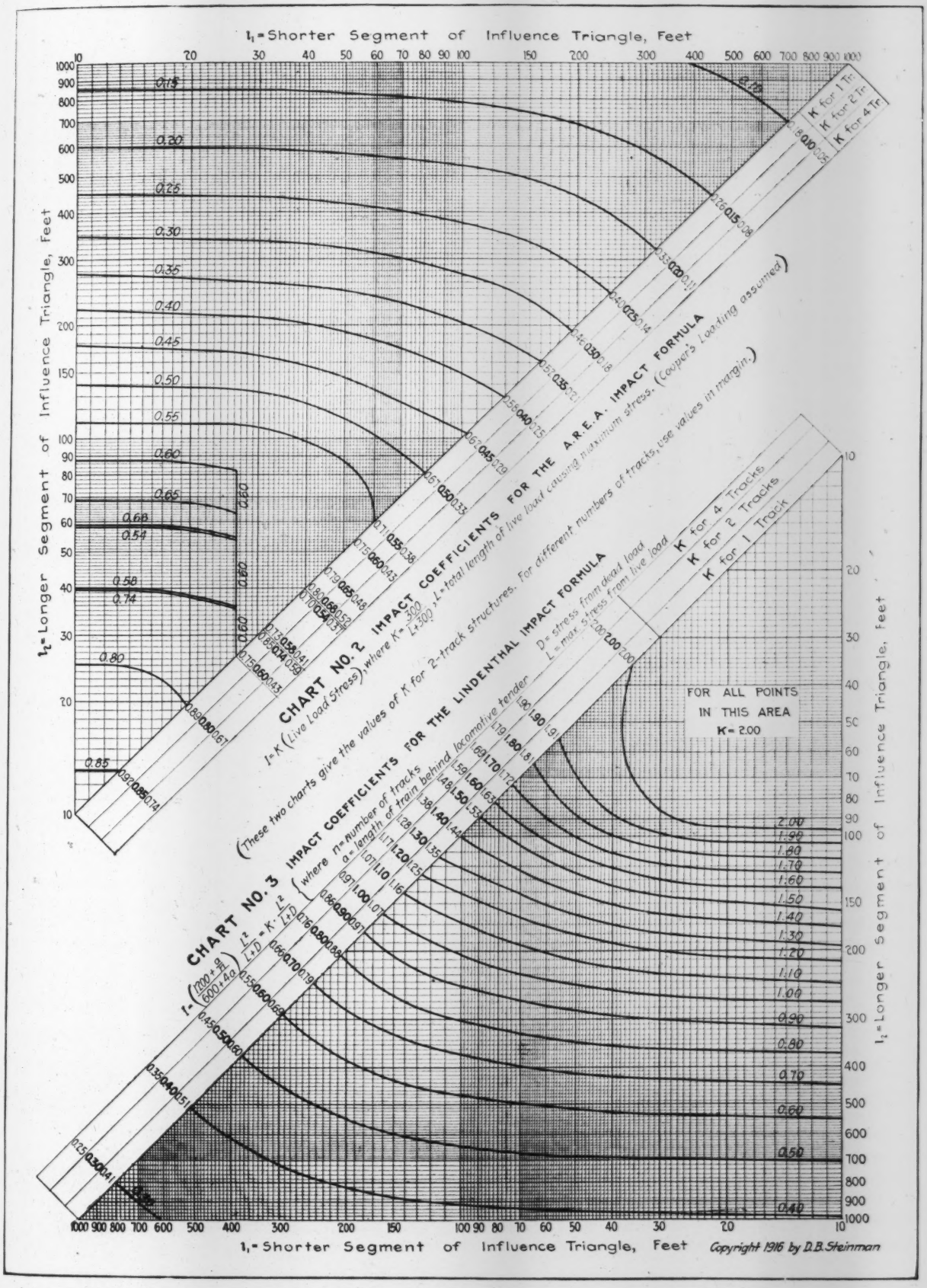
l₁ = Shorter Segment of Influence Triangle, Feet
l₂ = Longer Segment of Influence Triangle, Feet
K for 1 Tr.
K for 2 Tr.
K for 4 Tr.
CHART NO. 2 IMPACT COEFFICIENTS FOR THE A.R.E.A. IMPACT FORMULA
I = K (Live Load Stress), where K = 300 / L+300, L = total length of live load causing maximum stress. (Cooper's Loading assumed)
(These two charts give the values of K for 2-track structures. For different numbers of tracks, use values in margin.)
CHART NO. 3 IMPACT COEFFICIENTS FOR THE LINDENTHAL IMPACT FORMULA
I = (700 + 9L / 600 + 4D) L²/L+D = K · L²/L+D
(where n = number of tracks, a = length of train behind locomotive tender, D = stress from dead load, L = max. stress from live load)
K for 4 Tracks
K for 2 Tracks
K for 1 Track
FOR ALL POINTS IN THIS AREA K = 2.00
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In this case $l_1 = l_2 = 60$. By the writer's chart of equivalent uniform loads, $q = 7{,}700$ and the resulting bending moment per girder is

$$L = \tfrac{1}{4}\, q\, l_1 l_2 = 6{,}930{,}000.$$

The dead-load bending moment is

$$D = \frac{5{,}200}{7{,}700} L = 4{,}680{,}000$$

Using the A. R. E. A. impact chart, $K = 0.71$ and

$$I = KL = 4{,}920{,}000.$$

Hence the total bending moment is

$$D + L + I = 16{,}530{,}000.$$

Using the Lindenthal Impact Chart, $K = 1.86$ and

$$I = K \frac{L}{L + D} L = 7{,}700{,}000$$

Hence the total bending moment is

$$D + L + I = 19{,}310{,}000.$$

Although this is greater than the preceding result, the required section will be about the same since the permissible unit stress is 20,000 against 16,000 for the A. R. E. A. specifications.

Example 4—For one of the members of the Hell Gate arch (four-track), $L = 8{,}035{,}000$, $D = 16{,}462{,}000$, $l_1 = 489$ and $l_2 = 489$. To find the impact stress and required cross-section.

For the A. R. E. A. Impact Formula, Chart 2 gives $K = 0.07$. $I = KL = 562{,}000$.

$(D + L + I) \div 16{,}000 = 1{,}570$ sq.in. required.

For the Lindenthal Impact Formula, Chart 3 gives $K = 0.36$. $I = K \dfrac{L}{L + D} L = 950{,}000$.

$(D + L + I) \div 20{,}000 = 1{,}272$ sq.in. required.

For short spans, below 100-ft. for girders and 200-ft. for trusses, the Lindenthal specifications produce somewhat heavier structures than the A. R. E. A. specifications, the difference being principally in the web members. As the spans become longer, the Lindenthal specifications yield an increasing economy.

Concrete Viaduct Versus Fill for Street Elevation

Astoria, Ore., may be called a city on stilts. Situated on tidewater at the mouth of the Columbia River and for many years being without railroad connection, the city naturally grew up about its docks. With growth and expansion the boundaries reached to sidehill ground beyond the tide flats bordering the river, but the business district remained, built upon piling, close to the waterfront.

As Astoria has grown in importance and more substantial buildings have taken the place of the original frame structures, the disadvantages of building on piles elevated above the ground surface has become more apparent. With the development of hydraulic dredging it has been possible to fill in the tide flats with sand pumped from the river channel at a very reasonable cost. By the formation of improvement districts a large portion of the tide flats at Astoria has been filled in, either partly or to street grade.

There has recently been brought up for improvement a district that has been partly filled with dredged material, but in which the surface is still from 8 to 12 ft. below street grade. As an alternative to completing the fill there was suggested a concrete viaduct construction, which received considerable support from various quarters with the result that designs were made and approved by the city council.

Fig. 1 shows typical details of the first unit of viaduct construction, on which bids were called for. This was for the improvement of 11th St. from the railroad right-of-way near the waterfront to Exchange St., a total length of about 950 ft. As may be seen, the construction consists of four-post bents carried on timber piling, and a girder and slab deck. The width, curb to curb, is 34 ft., with 8-ft. cantilever sidewalks to the full street width of 50 ft. Temporary wooden walks were proposed for a part of the work, to be replaced later by concrete slabs. Expansion joints, with lead sliding surfaces, are provided at every third bent, or at 50-ft. centers.

The principal argument in favor of the concrete viaduct construction in the place of a solid fill was that it provided an excellent vault for public utilities, such as telephone, light and power circuits, water, gas and steam-heating pipes. To inclose this vault, it was proposed to build hollow-tile sidewalls between the outer lines of posts, or at the curb line.

The viaduct construction as outlined, while receiving the support of certain local interests in Astoria, was opposed by others, notably several large taxpayers who would be subject to assessment and taxation for the improvement. These taxpayers called upon engineers to make a study of the situation and report upon the relative economy of different forms of construction.

The costs deduced from this study are of particular interest, as they give light on a problem that is often

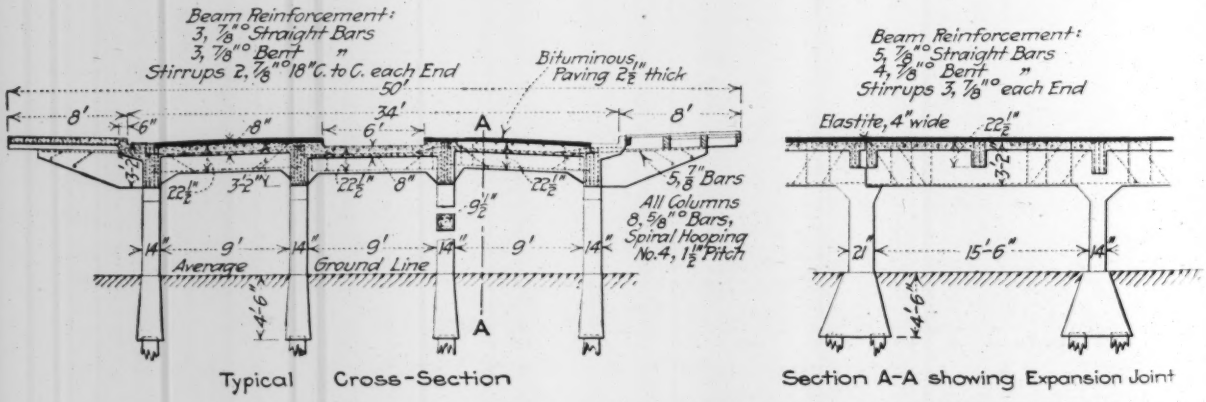
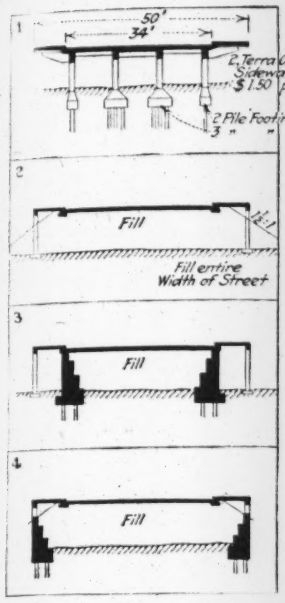


FIG. 1. DETAILS OF REINFORCED-CONCRETE ELEVATED STREET PROPOSED FOR ASTORIA, ORE.



	COST PER LINEAL FOOT		
	Without Sidewalks	With Concrete Sidewalks	With Timber Sidewalks
1	$ 55.60	$ 61.80	$ 57.00
2	$ 17.00	$ 25.30	$ 19.10
		$ 20.00	$ 19.40
3	$ 47.30	$ 55.60	$ 49.60
4	$ 39.80	$ 46.00	$ 41.70

FIG. 2. COMPARATIVE COSTS OF FOUR DIFFERENT TYPES OF ELEVATED STREET

met, in which fills and different types of wall construction are alternatives to open spans. Using the conditions at the proposed 11th St. viaduct in Astoria as standard, designs and estimates of cost were made upon three other forms of construction, comprising a sand-fill with slopes from the street line and two types of retaining-wall construction—one with the wall at the street line and one at the curb line. The alternate designs, together with the summarized costs of each, are given in Fig. 2. For comparison with conditions existing elsewhere the unit costs of materials considered in these estimates may be of value. They are for date of Apr. 13, 1916:

Cement, per barre	$2.00
Sand and gravel, per cubic yard	1.00
Reinforcement, per 100 pounds	4.50
Piling in place, per lineal foot	.20
Excavation, per cubic yard	.50
Paving, per square yard	.75
Lumber in place, per thousand	25.00
Sand-fill, per cubic yard	.20

It will be noticed that the estimated cost of the sand-fill, complete with walks and paving, is $19.40 per lin.ft., against $57 for the viaduct scheme. This comparison was brought out very clearly in the final discussion of the improvement before the city council of Astoria. The figures were confirmed by the testimony of several engineers. In spite of the strong showing made against the viaduct improvement on the ground of cost, the city council approved it and authorized a call for bids.

For surfacing the street upon the viaduct there was specified a patented bituminous paving material. As the use of this material, requiring license from the patentee, would to some extent interfere with free bidding upon the entire improvement, just previous to receiving bids the paving contract was separated from the viaduct.

Subsequent to the opening of bids and before the award of the contract for the viaduct improvement, certain large taxpayers of Astoria brought injunction proceedings, based upon legal technicalities. As the following out of these cases would have delayed all work for a considerable time, the city council of Astoria subsequently agreed to reject all bids and readvertise for a sand-fill.

Concrete Piles 100 Ft. Long

Precast reinforced-concrete piles 100 ft. in length have been used in the construction of wharves at Auckland, New Zealand, and were described in a recent number of *Concrete and Construction Engineering* of London, by F. E. Powell. These piles are the longest concrete piles ever driven.

One of the wharves, about 281x1,200 ft. in plan, required nearly 2,000 piles, and borings showed that in some places it was over 100 ft. from deck level to rock bottom, with some 50 ft. mud and clay overlying the bottom. It was felt that unless the piles were driven to refusal on the rock, there must be some doubt as to the continuity of the girders and beams, all of which were designed as continuous over the supports. Although reinforced-concrete piles 70 ft. long and 18 in. square had been employed in the work, these piles were hollow and light and their handling had not involved any particular difficulty. The difficulty with the longer piles was, of course, in their handling, and a scheme for swinging them into the leads of the piledriver had to be worked out.

Fig. 1 shows a section of the piles and the method of slinging them. The pile as shown there is 20 in. square and 100 ft. long, reinforced with eight longitudinal rods. Near the two ends and at about the third point, cross pipes 4 in. in diameter were put through the pile section to carry 3½-in. pins, to which the slings were attached. The pile is then carried by a wire rope made fast at its ends and running free through the three-sheave block and two snatch blocks secured to the pile, which had to be up-ended and controlled by other ropes until in a vertical position alongside the piledriver leads. In discussing the design of the sling, Mr. Powell says:

> Although the scheme was simple in this form, a large amount of work was necessary before it could be put into execution. Fortunately, the Board had an 80-ton floating steam crane which could be used to assist operations. Then the calculation of the proper points of support had to be made to suit the safe moments of resistance of the piles, having regard to the fact that the rope has equal stress throughout its length, being free to run in all the pulleys, while its angles of inclination to the pile vary with the slope of the pile. Bending moment diagrams were, therefore, made for each point of support over the range of movement and various positions tried until the worst bending moments were always within safe limits. The section of the pile shown was adopted as a trial and found to meet the case. This section was used for all the piles from 60- to 100-ft. lengths, the shorter ones being slung from two points only.

Under the arangement devised it was necessary to have a diver remove the 3½-in. pin at the lower end just prior to diving. When the first pile was slung into place it was found that the three sheave blocks shown in the main part of the diagram jammed badly under the

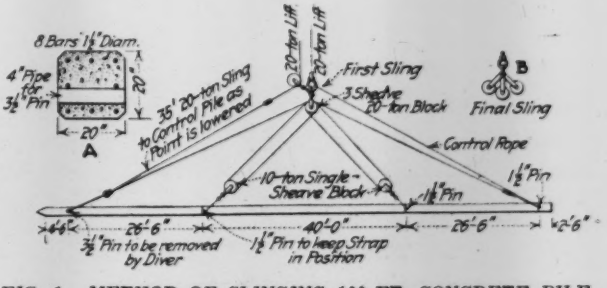


FIG. 1. METHOD OF SLINGING 100-FT. CONCRETE PILE AT AUCKLAND, NEW ZEALAND

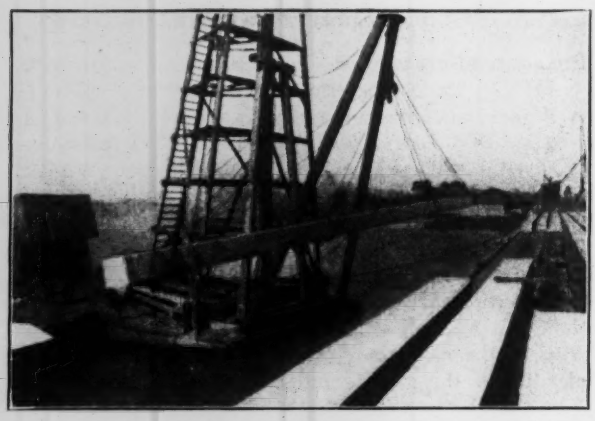


FIG. 2. LIFTING ONE OF THE NEW ZEALAND 100-FT. CONCRETE PILES INTO PLACE

load and a block of the type shown separately in the diagram having the sheaves all in the same plane was made. The piledriver available would not stand up under the work so a new machine had to be built having leads 94 ft. high, with a five-ton hammer.

Deflection of the piles during lifting was very small. The central portion remained practically straight and the maximum deflection at either end about 1¼ in.

A large number of these piles varying from 50 to 100 ft. in length, and 12 to 20 tons in weight, were subsequently made and driven. Only one was broken in the operations. The piles were mostly driven home to the estimated depth but only by long continued hammering. The accompanying view shows one of the piles being swung to the lead in the piledriver.

Large Wheel-Type Excavator

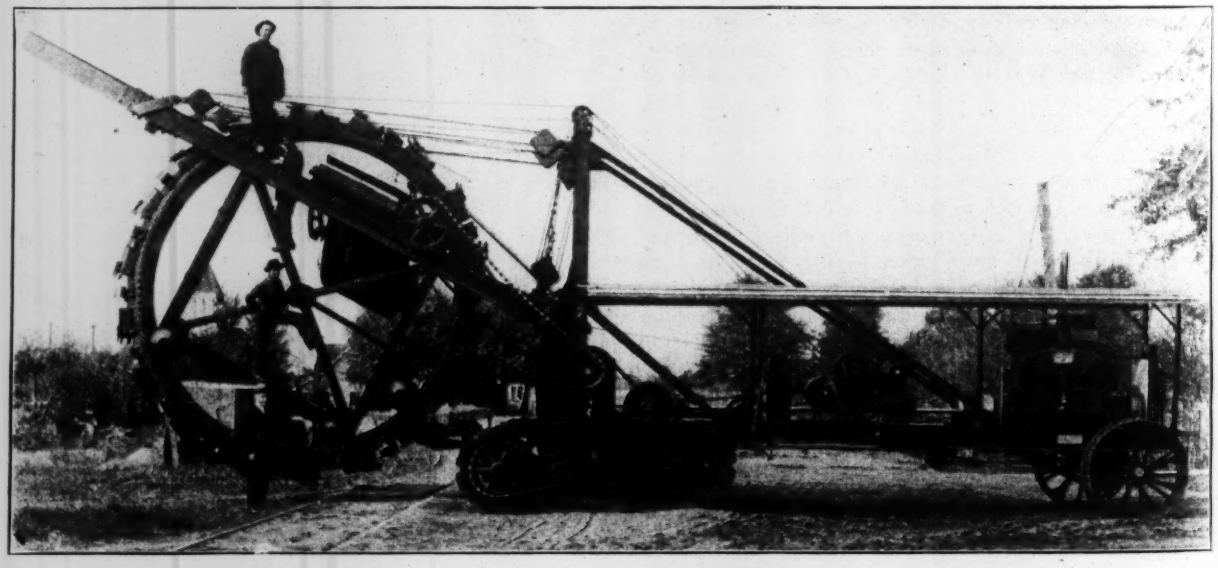
The large trenching machine shown in the accompanying view can dig trenches 42 in. wide to a depth of 12 ft. It is owned by Thornton Bros. Co., contractors, of St. Paul, Minn., and has been used in heavy trenching for sewers and other municipal work. Similar machines of various sizes, and for use in hard and soft ground, are used in ditching and trenching for land drainage, irrigation, pipe lines and municipal work.

The excavating wheel of the machine illustrated is 13 ft. outside diameter and 14½ ft. inside diameter. It has no spokes or hub, but its internal face rides upon steel wheels carried by a rectangular frame attached to the pivoted boom. It has an internal gear on each side, driven by pinions on the boom, with chain drive from the engine on the main frame. The wheel is fitted with "contractors'" toothed buckets for excavating hard material. Alternate buckets have heavy projecting teeth or rooters on the sides to assist the cutting edges of the buckets in trimming the sides of the trench. The buckets deliver their contents upon a broad conveyor belt which discharges the material upon the spoil bank. The outer end of the hinged conveyor frame has a bail with supporting tackle led to a light pipe boom pivoted to the headframe. The conveyor has a friction drive.

The boom consists of two steel channels (between which the wheel works), and has its heel sliding in guides in the vertical head frame. This boom is supported by tackle at each end. Two sets of tackle attached near the outer end of the boom provide for swinging it vertically upon its heel pivot or hinge. Another set is attached to the pivoted end and provides for raising and lowering the heel of the boom in the headframe.

With the boom shifting vertically in this way, there would be variation in the slack of the chains driving the excavator wheel. To provide for this, and thus prevent the chains from fouling the sprockets, a compensating device is required. For this purpose, the lower part of the headframe has curved guides for the heel of the boom, the curve being such that the distance between the driving and driven sprockets is maintained practically uniform. In the smaller machines, the slack is taken care of by idlers on the wheel frame.

The main hoist is at about the middle of the machine. The frame is supported on a pair of wheels at the forward end, and by caterpillars at the rear end. Each caterpillar has a track 3 ft. wide and is 8½ ft. long between the sprocket shafts. One of these shafts has a chain drive, as shown. Upon the frame is a four-cylinder



WHEEL-TYPE EXCAVATOR FOR CUTTING TRENCHES TO A DEPTH OF 12 FT.

100-hp. gasoline engine, furnishing power for the several hoists and for the caterpillars. The road speed is about 1 mi. per hour.

The machine is 52 ft. long and 13 ft. wide over all, with a height of 22 ft. to the top of the headframe, and a working weight of about 43 tons. It was built by the Buckeye Traction Ditcher Co., of Findlay, Ohio.

The machine illustrated has been used by the Thornton Brothers Co., of St. Paul, Minn., for excavating trenches 12½ ft. deep and from 28 to 42 in. wide. Under fair conditions it could dig 700 to 900 ft. of trench per day. The material excavated consisted of 3 ft. of clay, with sand and gravel below. Before this became frozen the sides were allowed to slope, digging down about 8 to 9 ft. deep, which gave a ditch practically 14 to 16 ft. wide on top and narrowing toward the bottom. Afterward the ground froze from 4 to 6 ft. deep, and fires were built ahead of the machine (sometimes requiring a double fire). These thawed the ground approximately 3 ft. deep, and the balance of the frozen material was cut with the excavator. In this way making a cut 7 to 9 ft. deep, the machine dug from 300 to 500 ft. per day.

How to Appraise Water Rights*

By Harry Barker†

SYNOPSIS—It is attempted here to set forth, in place of the confusion now existing among valuation engineers, a logical, sound and reasonable procedure which will be generally acceptable for the valuation of vested rights to use water. Water rights are considered as having fundamentally the nature of industrial real estate and to be appraised on the basis of earning power. To assist in the comparison of water rights of determined and of unknown values, the value of a right is considered as having two factors—a regional and a constructional value. Limitations to the use of steam-power comparisons are noted. Difficulty shown of appraising water rights in rate cases. True value of storage-reservoir sites set forth.

The large amount of water-power valuation being made all over the country, the peculiar results being presented by some well-meaning but uninstructed appraisers and the hesitation of commissions to accept the results make timely a review of defensible procedure in water-rights appraisal. Valuations that can be criticized are those, for example, which set forth the worth of water rights as the "combined intangible and going values of a power development"—locating this as the capitalized possible net annual income. Such a finding has no justification for parading as the value of water rights, although water rights indeed are hidden therein.

Water Rights Must Be Considered

Greatly varying ideas are prevalent as to whether or not a public utility utilizing a water power can include in the rate-basis worth of its plant any allowance for rights to the beneficial use of water.

This has been denied in a few specific cases, notably by the Public Service Commission for the Second District of New York in Fuhrman vs. Cataract Power and Conduit Co. (3 N. Y. Pub. Ser. Comm. 2nd D., 670) on the grounds that the Federal and state permission to use water at Niagara Falls gave the corporation no title to the water and hence no water right measured by the price of steam power. Other commissions, for instance, like those of California (*re* Northern California Power Co., Calif. R.R. Comm. Rep., 1913) and Idaho (*re* Pocatello Water Co. 1 Idaho Pub. Serv. Comm. Orders 78; 1914) appear to have held at one time that water rights were in the nature of fictitious intangibles and so not admissible as part of rate-basis worth.

Later ideas of both these bodies named (see *re* City of Santa Cruz, Calif. R.R. Comm. 2,666; P. U. R. 1915 F 768, and *re* Pocatello Water Co., Idaho Supreme Court, 150 Pac., 47; P. U. R. 1915 F. 437) show a different attitude, for in both cases a value for the rights was allowed.

There has existed little doubt anywhere as to the necessity of giving proper attention to water rights since the United States Supreme Court held in 1914, in San Joaquin and Kings River Canal and Irrigation Co. vs. Stanislaus County (233 U. S., 459), that in spite of a declaration in the California constitution that water appropriated for sale was appropriated for public use, the benefit was private and the rights thereto should be considered in condemnation and rate cases. Mr. Justice Holmes thus speaks:

> But it is said that as the plaintiff appropriates this water to distribution and sale, it thereby dedicates it to public use under California law, and so loses its private right in the same. It appears to us that when the cases cited for this proposition are pressed to the conclusion reached in the present case, they are misapplied. No doubt it is true that such an appropriation and use of the water entitles those within reach of it to demand the use of a reasonable share on payment. It well may be true that if the waters were taken for a superior use by eminent domain those whose lands were irrigated would be compensated for the loss. But even if the rate paid is not to be determined as upon a purchase of water from the plantiff, still, at the lowest, the plaintiff has the sole right to furnish this water, the owner of the irrigated lands cannot get it except through the plaintiff's help, and it would be unjust not to take that fact into account in fixing the rates. . . . It seems unreasonable to suppose that the constitution meant that if a party, instead of using the water on his own land, as he may, sees fit to distribute it to others, he loses the rights that he has bought or lawfully acquired.

This case is widely quoted as controlling. The question now is not whether any water right is to enter rate basis, but how much should be allowed. The hesitant attitude of public representatives as to amount is well expressed by the New Hampshire Public Service Commission when it states (*re* Grafton Electric Light and Power Co., 4 N. H. Pub. Serv. Comm. Rep. 178):

> The objection of the Supreme Court to the conjectural character of the cost of reproduction method [of valuing property in the Minnesota Rate Cases] applies with equal force to

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the "saving-over-coal" method of valuing water power. It assumes, what is not proved, that power could be produced profitably by coal, and it assumes, what is not true, that a given amount of power produced by water varying in amount, as it will on even the best-regulated streams, is equal in value to a like amount of power generated by steam, constant and reliable at all times.

We live in a region remote from the coal fields, the cost of transportation is heavy, and the price of coal is higher than in almost any other part of the country. On the other hand, ours is a mountainous state with many streams having a large fall and furnishing an abundance of water power, much of which is still undeveloped. . . . A fair value of a water power in New Hampshire cannot be a value which takes no account of our natural resources and makes electricity produced by water as expensive to the public as if produced by coal.

The Vermont Public Service Commission also expressed the same hesitancy in *re* Montpelier & Barre Light and Power Co. (No. 452, 1916; P. U. R. 1916 B 973), as follows:

The commission, however, entirely disapproves of this method of determining the value of water rights [by direct comparison with steam] because, in our opinion, it creates a value which may be largely in excess of the actual value of the rights and because this method, if applied to rates, would entirely deprive the consumer of any benefits to which he is entitled by reason of having these natural resources at hand.

Rights as Real Estate

At the outset it should be stated that authorities are universally agreed that water rights are property having essentially the nature of real estate—industrial real estate. Therefore they are valuable in accordance with their power to produce income.

The Idaho commission in valuing the water rights of a hydro-electric company (Taylor vs. N. W. Light and Water Co.; Idaho Pub. Util. Comm. Order 297; P. U. R. 1916 A 372) refused to allow a value where it transpired that the company could purchase cheaper than it could develop.

But this value of earning power must be separated from the common intangible values also generally recognized by their earning power or by investment. Even where such procedure is followed, the greatest variation is seen in the results of its use by different persons. The appraiser with sympathies running in one direction seems apt to make value large enough to compensate the holder for amounts more desired than supported by fact. The appraiser of opposite sympathies seems to minimize the value of water rights where these can be construed as depending in any measure on public grants. As is usual on such controversial matters, the actual truth in any specific case generally resides in a somewhat indefinite middle ground and can be approximated only by the exercise of good judgment as well as the study of specific data.

If water rights are essentially of the nature of real estate, it would be expected that they should be valued like real estate by the direct comparison of sales. However, this method, in simple form, is extremely restricted for the study of water-power rights; exchanges are extremely few in number compared with the parcels of land that are traded every year. Moreover, in comparing the values of real estate it is essential to take into account transfers of land in the immediate vicinity of the parcels studied. That seldom is possible in the case of water powers, and in making any direct comparison of sales the geographical separation usually will be sufficient seriously to impair the validity of the appraisal—unless such effects as different cost of development, different annual return, different market conditions and different probable net profit are brought into play. But the handicap of geographical separation may be overcome by scientific analysis preceding the attempt at comparison.

Splitting the Value of Rights

Water rights, it has been stated already, exhibit the peculiarities of *industrial* real estate in that the value depends on utility for special service. Indeed, consolidated water rights may be regarded as the enhancement of value of land well situated for the special use of generating power from falling water, etc. The enhancement from these special uses is far greater than the value of the real estate for all the ordinary uses of land, so that the common value of the tracts is generally neglected as inconsequential.

This enhancement consists of two elements of value, the first depending on the physical conditions which lead to ease or difficulty (measured by expense) of developing the rights. The second depends on the demand for water or power in the locality, and the kind and cost of power that predominates to rule the market. The two values are not wholly independent, for where there is competition with steam power, the second element increases with any decrease in cost of development.

These value factors we may call[1] "constructional" and "regional." In case either factor is zero, the value of the water rights is zero. For instance, the many water powers hundreds of miles back from even moderately populous areas have little or no regional value at present, and the value of the water rights is inappreciable in spite of possible high constructional value. The numerous water powers of the Adirondack region of New York have no local market and cannot be completely developed to compete profitably with steam power in the metropolitan markets; the regional value is low. If it were sought to dam the Hudson River and develop power anywhere near New York City, the regional value would be great, but the constructional value obviously would be zero, so great would be the legal, engineering and financial obstacles.

Comparing Two Rights

If an engineer has data at hand on the sale prices of scattered water rights, the unknown value of a questioned private water right can be approximated by increasing or decreasing the known value in proportion to (1) the size of the possible development, (2) the difference in regional value and (3) the difference in constructional value. Much the same sort of scheme would be employed by an experienced engineer, the rights of actual scattered plants taken before for comparison being replaced by those of an ideal normal plant existing in the mind of the appraiser. It might be thought that the problem could be approached also by setting up for comparison a hypothetical plant of the specific local capacity, built under local hydraulic data but under fixed difficulties and at specific cost, and finally having a specific market and specific income. For such a plant there could be computed a net income and the water rights would depend on this—being in the nature of real estate which is valued by its earning power. Then the real water rights could be approximated by changing

[1]Following the nomenclature employed by Robert E. Horton, Consulting Engineer, Albany, N. Y., in recent cases.

this value to accord with greater or less construction difficulties which developed, and for more or less favorable market conditions which arose. But this last study reduces more or less to appraisal by earning capacity.

Valuation of Rights by Earning Capacity

Where there are no dependable data for the method of appraisal by sales comparison, it is necessary to fall back upon an earning-capacity study. At the outset it is necessary to start with the query: What sum could an intending purchaser afford to pay for consolidated water rights going with a diversion for power or water-supply, knowing the gross income which the power or water will bring, knowing that certain fixed charges, interest and taxes must be paid on the full investment—including land and water rights and the cost of plant and business development? All the factors entering into the solution of the problem are known but one—water rights—and the answer is therefore ascertainable.

On the basis of earning capacity, the capitalized net profit is equal to the cost of the physical plant and going-concern value (the latter estimated for an undeveloped business by any of the acceptable methods) plus the water rights in question. The net earnings, or profit, in turn are the gross annual earnings less the annual operating cost, depreciation compensation, supervision, repairs, etc., and less the interest and taxes on plant (structural) value, going-concern value and worth of water rights. This is briefly and simply stated[2] as:

$$S + G + W = \frac{P}{N} = \frac{I - O - (S + G + W)(R + T)}{N}$$

whence

$$W = \frac{I - O - (S + G)(N + R + T)}{N + R + T}$$

where P = annual net profits, W = water-rights value, I = annual gross income, O = annual operating expenses, S = structural cost, G = going-concern cost, N = capitalization rate, R = fair return on investment, T = tax rate.

The warning that needs to be made in the use of the earning-value method is that appraisers have seemed apt to under-estimate the annual operating expenses and so over-estimate the profits and the value of water rights.

Short-Cut Steam-Power Comparison

The gross annual income of a water-power plant in the East is tremendously affected by the cost of steam power. There are few hydro-electric plants where a considerable proportion of the output does not compete with steam drive directly. Even where the service is for lighting, the price is limited to that of possible local steam-plant current. Therefore, there has been used in the East a short-cut scheme of appraising water rights by capitalizing the difference in cost of output between the hydraulic plant and an equivalent steam plant in the same district. This may be permissible at times, but it is not to be universally recommended as it substitutes an extreme hypothetical condition where a closer approach to actuality is possible. Public hostility to it has already been noted. For instance, with utility regulation, even in the East, the tendency of rates is toward cost of service, including therein a reasonable return on the property employed. The regulated rates are not directly or fully related to the cost of steam power. (The off-peak factory-drive competitive business, however, is secured only by cutting under steam-power costs. The off-peak commercial business operates to reduce the general cost of service by giving more constant utilization of the plant and diluting the overhead expenses with small additional profits.)

The desire to use steam-power costs in the valuation of water rights springs out of the convenience of such comparisons rather than rising from their applicability. The capitalized annual cost of operating and maintaining in perpetuity a profitable steam plant to produce an amount of energy equal to that which can be produced at a given water power generally will prove larger than the similar figure for a water-power development, the water rights of which are determined by weighted comparison of sales or by study of actual earning power. Therefore the value of water rights determined by such steam-power cost comparisons may be expected to be often large enough to prove an embarrassment to the owners in time of protracted business stagnation, financial depression, strikes, war, unfavorable legislation, strict regulation or poor internal management. When this scheme is or has been employed, care should be exercised to see that the valuation of water rights and plant plus capitalized operation and maintenance costs does not exceed the capitalized sum which would maintain and operate the equivalent steam plant *not continuously, but for such portion of the time as the water-power plant could reasonably be expected to operate under the unfortunate contingencies noted.* The necessity for such a scrutiny may be realized when it is recalled that the investment costs, which continue always, are high for the water-power plant and low for the steam plant; the operating expenses which may be curtailed during depressions are small for water-power and large for steam.

Value of Rights Under Regulation

Many believe that the valuation of water rights in the Western states must be on a different plane from that in the East. There is nothing seen, however, in the sales-comparisons or earning-value plans of appraisal but what can be followed in one section as in another.

Some publicists claim that as the water rights came from the public and are used for the public good, no value should be allowed the utility company for those rights. The actual conditions under which public grant of water powers has been made need to be considered, of course. It is conceivable that in some cases the public desired that the company should earn on its investment and activity, but not on the fundamental land and water rights. Where that view actually prevails, obviously there can be no water right owned by the utility company. Where such a condition has not been found, the appraiser has to find earnings even under rate regulation. This is difficult, for he has to avoid a reasoning-in-circle situation. That is, if he takes rights through rates based on a worth which includes value of water rights, the larger the value of those rights the higher the rates must be; then the larger becomes the value of the rights again and the higher the rates, and so on. The only escape usually is to consider what the water rights would be in private service.

The concern may have obligated itself to serve the public when it might have continued to give private

[2] Following a form suggested by Robert E. Horton.

service in large blocks; then the water rights ordinarily should be considered as before the public service became predominant. Surely neither the company nor the public expected that the company would be penalized for furnishing a multitude of users instead of the few. The right to use water for private gain of course is as legitimate as the right so to use land. Care has to be taken to insure that the estimate of earnings on a private-business basis is strictly consistent, that it does not include income-rate enhancement promoted by public assistance marking the change from private to public utility—like occupancy of streets or of power of eminent domain.

There are various ways away from reasoning in a circle when the very earnings to be taken in the earning-power method are under attack as unreasonable. First, it must be ascertained what peculiar status any Federal and state legislation may give the water rights; or what specific franchise contracts may affect them so that they may be declared nonexistent, existent but partly dedicated to public benefit, or existent and entirely in private possession.

If existent and in private seizure, in most cases there appears to be no escape from the higher prices due to earnings on water right. A parallel case exists: A state may have given away its valuable mineral land to any who would develop, and it cannot but endure the situation as to old land patents, though it can demand a share of the profits in land yet ungranted. It is the same way with water rights in many cases. Legislatures may have the right to cancel the rights by repayment.

The actual estimation of probable earnings of the rights under purely private industry may be arrived at in some cases, for instance, from the figures available from the private utilization of part of the water rights as leased to others.

The "value" obtained in the ways and under the conditions noted is in reality more than simply worth of water rights. It has been noted that it includes, first of all, the relatively unimportant value of the appurtenant land for common uses. If the plant is already built and in operation, the value may often also include a factor depending on the brains worked into the design, construction and operation.

The separation of the value of the real estate for building or agricultural purposes may be carried out with reasonable simplicity, but the value of applied mentality is very difficult to separate from the water rights. Therefore it is common to allow all of these accompanying values to remain consolidated with the value of water rights, and no case is recorded of resulting injury. Care must be exercised not to admit the same application of mentality in two separate parts of a utility valuation.

A Water-Rights Fallacy

In actual cases there are marked differences in the opinions and procedure of the different sides. For instance, in condemnation proceedings the owners frequently compute the annual cost of their power so as to include the interest and taxes on only structural investment—purposely neglecting charges on water rights. They then subtract this cost of their water power from the annual cost of some substituted form of power, like steam, and arrive at an apparent annual profit which they capitalize at 8, 6, 5 or 4 per cent. The resultant figure is their contended value of water rights. This procedure is defended on the claim that since they own the water power they are not paying interest on the rights, and it should not be charged against them in comparing the cost of water power *to them* with the cost *to them* of some substitute power.

Obviously such a procedure is most unsound. If the water power were worth all that is shown by such a series of computations, the plant and rights could be sold and the money invested in Government bonds to bring by pure interest an annual profit as great as could be secured after undergoing the risks of development.

On the other side of such condemnation cases, the would-be seizors claim that the value of water rights cannot exceed a sum which a purchaser could afford to pay if he borrowed all the funds for acquisition of rights and development of plant, and if the enjoyment of the rights enabled him to derive a small safe profit in return for service in promotion and management and as a guarantee of permanent ability of the works to afford enough revenue to cover interest and depreciation indemnity.

Valuation of Storage Reservoirs

A problem now becoming pressing is the valuation of storage-reservoir sites. In a few cases the estimated or experienced total annual gross return from the increase in available power over that of no storage has been capitalized to secure this value (with or without deducting investment charges on the reservoir) on the evident assumption that there would be no increase in size of generating plant or in the operating expenses. It is more conservative, but not wholly satisfying, to capitalize the net annual earnings from the increase in power. A still more rational procedure would be to compare the entire situations with and without storage; then the value of the storage site might be, as a maximum, the capitalized difference in net earnings of the larger works with storage (deducting all operating costs and fixed charges on generating station and storage works complete) and of the smaller plant that would have been built if storage had been impossible. In securing these net earnings it hardly need be said that recognition must be given to the amounts of primary, permanent, assured or continuous power and of secondary, nonpermanent, unassured or noncontinuous power produced under the two conditions of storage or no storage.

Where one reservoir benefits a string of plants scattered downstream, it is common to state that the value of the storage depends on the amount of fall through which the impounded waters may be utilized. As a quantitative statement this may be inaccurate, for the general benefits of storage decrease with the distance downstream. This is due, first, to the fact that the impounded waters form a smaller and smaller proportion of the stream flow as added units of drainage area pour their runoff into the stream. It is due also to the disturbing influence of local-plant pondages below, especially where they do not operate alike at simultaneous hours or where they are not under the control of a single load dispatcher. The value of the reservoir site then is not the sum of capitalized net earnings of each plant assumed to secure full benefit of the stored water; it would depend upon the somewhat complex and difficultly ascertained difference in net earnings for the entire existing or probable situation of all the plants as a unified system with and without the storage.

Canadian Society Fixes Fees for Engineering Service

The Association of the Alumni of the Ecole Polytechnique of Montreal (Anciens Elèves de l'Ecole Polytechnique de Montréal) has issued a code of professional ethics and list of fees. The code sets down the usual ethical considerations, distinguishes between a "consulting engineer" and an "expert engineer" and outlines their respective duties. The most interesting part of the paper is the tabulation of fees. These are as follows:

For simple consultation the charge to the client will be proportioned to the time spent on the work, the basis of the charge to be a day of 6 hr., remunerated at a rate of $50 to $100 a day, depending upon the importance and experience of the engineer. If his experience and reputation justify it, a larger charge may be made. For fractions of a day, charges are as follows: $30 to $60 for a half-day; for a lesser portion, $15 to $25 per hr.

For a written report the following charges are to be made: For preliminary study of a project with a report, or for an examination of a project prepared by another engineer, with a report on the subject; for a study and report on questions of litigation; for preparation of depositions before courts; presence in courts during trial—$50 to $100 a day or more for the first five days devoted to the client and $25 to $50 a day for the next succeeding days. The client shall in addition pay all the salaries of assistants and an additional amount equaling 100% of those salaries to cover the overhead expenses of the engineer's office.

The working day of an engineer shall be 6 hr.; and when he is absent from his home city, his charges per day shall be based on the charges for the first five days' work as previously noted, and every day of 24 hr. or fraction of such day is to be considered as a day to be paid for without regard to whether the time is actually devoted to the study of the project in hand.

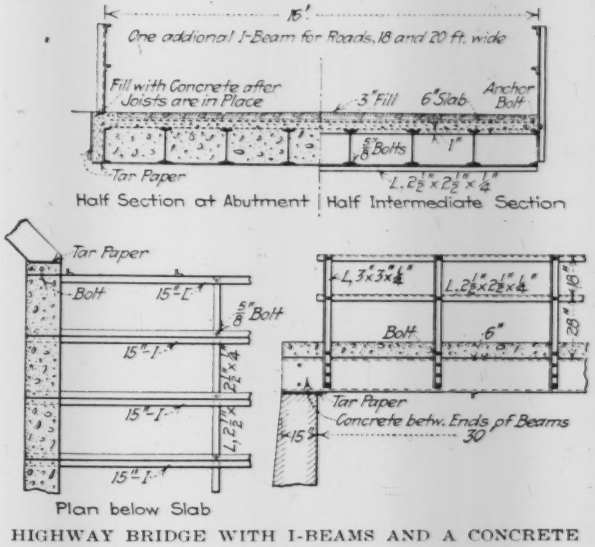
In some cases a consulting engineer is justified in charging a percentage of the actual cost of the enterprise under consideration. On such a basis the following charges may be made: For the preparation of a project, or the examination of a project prepared by another engineer, charges for services of the engineer and of his personnel shall be equal to 2% of the possible cost of the enterprise, if this total not more than $50,000 or less than $10,000. For all projects of probable cost of less than $10,000 the remuneration shall be at least $200. For projects of $70,000 to $100,000 the percentage shall be 1½% of the cost; for projects of more than $150,000 the percentage shall be 1% of the cost.

For preparation of designs for construction the fee of the consulting engineer and his personnel shall be as follows: 3% of the cost of the work, if $50,000 or less, with a minimum fee of $300 for work under $10,000. For projects of more than $70,000 and less than $1,000,000 a fee of 2½%; and for projects of more than $1,000,000, 2%. When superintendence of the work is included, the following charges should be made: Partial superintendence, comprising consulting work in the engineer's home office and some visits to the work during its execution, from 3 to 2% of the cost of the work. Complete supervision, comprising office consultations, visits to the work and superintendence of the execution by a representative of the engineer, control of the contractors, etc., from 5 to 7%, 5% applying to work of $1,000,000 or more, 6% to from $100,000 to $500,000, 7% to under $60,000. The minimum fee shall be $700.

Steel I-Beams and Concrete Highway Bridges

Highway bridges with I-beams resting on concrete abutments and carrying a concrete slab floor are used extensively in Iowa. The accompanying drawing represents one of a number of such bridges designed by John A. Illeman, County Highway Engineer of Boone County, Iowa.

The design covers spans from 16 to 35 ft., using 9-in. 21-lb. beams for the former and 15-in. 42-lb. beams for the latter. There are six beams for 16-ft. roads and seven for 18- and 20-ft. roads. In all cases the sides of the roadway are carried by channels of the same depth as the I-beams. Two transverse angles are bolted to the



HIGHWAY BRIDGE WITH I-BEAMS AND A CONCRETE DECK

bottom flanges of the beams and channels. Vertical angles riveted to the channels (and bonded to the concrete floor by long bolts) form the posts of the angle-iron hand-railing.

The deck is a concrete slab 6 in. thick, with its lower face 1 in. below the tops of the beams. It is reinforced by ½-in. transverse rods in the top and bottom, spaced 12 in. c. to c. (staggered) and ½-in. longitudinal bars, there being one longitudinal bar in the space between each pair of beams. The edges of the slab are 9 in. thick, forming curbs to retain a 3-in. fill of road material (usually gravel). Weep holes of 4-in. tile are provided for drainage, spaced 6 in. from the curb and 10 ft. c. to c.

At the abutments three layers of tar paper are placed on the bridge seats under the ends of the beams, and concrete is filled in between the ends of the beams, as shown. Tar paper is placed between this concrete filling and the wing-wall.

The bridges are designed to carry a 15-ton traction engine, with 11-ft. wheelbase and driving wheels (22 in. wide) spaced 6 ft. c. to c. The load is assumed to be 10,000 lb. on each driving wheel and 5,000 lb. on each front wheel.

Long-Span Howe Trusses Fail.

An unusually heavy fall of snow, most of it drifted on one side of the roof, caused the complete failure of a number of 110-ft. Howe trusses designed to support the roof of the Majestic dance hall, Salt Lake City, Utah. The accident occurred immediately after the snow storm of Mar. 25. It happened that no one was in the building at the time, although the very large hall (about 250x150 ft.) was regularly used for roller skating and dancing.

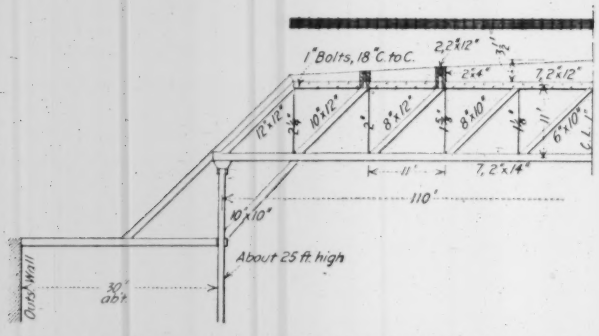


FIG. 1. SKETCH ELEVATION OF HOWE ROOF TRUSS, 110-FT. SPAN, WHICH COLLAPSED

The building was a frame structure completed only about two years ago. The main part of the hall was about 200 ft. long, roofed over by a series of 10-panel timber Howe trusses of 110-ft. span, spaced 16 ft. c. to c. The trusses rested on 10x10-in. posts, and carried the roofing by purlins at the panel points, supporting secondary rafters. The outside purlins rested on the upper chords and the center purlins on posts which raised the roof about 3½ ft. above the top chord.

The architect, B. O. Mecklinberg, Salt Lake City, declined to allow the details of the truss to be published, but the accompanying sketch, Fig. 1, gives the essential elements, based on the measurements and observations of those in charge of the wrecking and reconstruction of the building. The view, Fig. 2, taken by a local photographer immediately after the collapse, shows a truss at the east end of the hall which did not collapse.

The trusses as already noted were of 110-ft. span in 10 panels of 11 ft. each, and were 11 ft. deep, center to center of chords. The lower chord was built of seven 2x14-in. planks and the upper chord of seven 2x12-in. planks bolted together by 1-in. bolts spaced 18 in. c. to c. and staggered. The planks composing the chords were Oregon fir, surfaced on both sides and edges, so that their actual dimensions were not more than 1⅝x13⅝ in. and 1⅝x11⅝ in. In computing the cross-sections of the chords 1½x13½ in. and 1½x11½ in. were the dimensions used.

The diagonals were 12x12 in. at the ends, 10x12, 8x12, 8x10 and 6x10 in. from ends toward the center respectively. These were solid members mortised into the chords, without the use of castings at the panel points. The vertical rods were 2¼, 2, 1⅝, 1⅛ and 1 in. respectively from the ends toward the center. The roof purlins were two 2x12's supported by two 2x4's nailed or spiked to either side of the top chord.

As to lateral bracing between the trusses many conflicting statements are made. No design was made for this, and such as was put in is not a matter of record. Those in charge of clearing up the débris stated that there was no bracing whatsoever, and that the trusses depended entirely on the purlins, ceiling, rafters and bracing between the posts for lateral support. This seems hardly possible, yet there is no evidence of other bracing in the views taken immediately after the collapse.

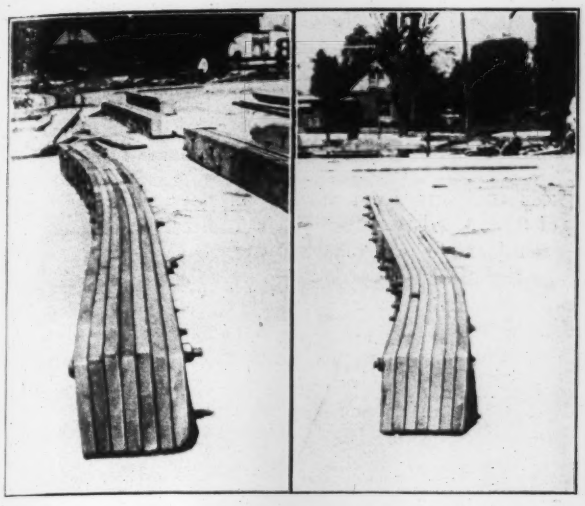
The failure was caused in almost every instance by the buckling and breaking of the top chord. Many of the posts on the north side of the building broke off below the kneebraces and the north wall fell outward as shown in Fig. 3. The bulk of the snow load was on the opposite or south side of the roof. The general impression seems to be that the initial failure was in the chords, although the posts must have been subjected to a considerable bending moment.

Those familiar with the condition of the trusses previous to the collapse believe that they were on the point of failure for a long time. Construction of the building was a rush job at the beginning of a season and everything was ahead of the plans. The timber was green, as

FIG. 2. EAST END OF BUILDING, SHOWING SINGLE REMAINING TRUSS AFTER COLLAPSE OF ROOF

nearly all available lumber is. It is said that ¼-and ½-in. shims were used in some of the mortised joints between chords and diagonals. In the top chord there were instances where the joints in the two outside planks came opposite each other. A section of such a chord was observed during the wrecking operations, May 9.

FIG. 3. NORTH SIDE OF BUILDING PUSHED OUTWARD



FIGS. 4 AND 5. VIEWS OF TOP-CHORD SECTIONS, SHOWING PERMANENT WARPS OR BENDS

When completed the trusses were ceiled off from the space below. Electric lights placed in the space between the ceiling and the roof for the purpose of showing the condition of the trusses were subsequently removed. The hall below was steam-heated, making this ceiled compartment above, containing the trusses, extremely hot and dry. Conditions were therefore ideal for a maximum amount of shrinking and warping. It appears that nothing was ever done in the way of tightening rods and bolts, and that long before the collapse occurred and during all the time the hall was in use many top chords were observed to be out of line by as much as 6 in., forming irregular wave-lines in ground plan. The condition of these top chords when the collapse did occur is shown clearly enough in Figs. 4 and 5, which are sections not more than two panels long. Moreover, several sections were warped also in the vertical plane. The amount of shrinkage in the length of the planks was plainly discernible in the wide openings at butting ends.

The designer was aware of these conditions and urged the owner (it is claimed) to bring the top chords back into line with turnbuckles and have them properly braced. In the meantime the holding company went through bankruptcy proceedings and the building changed hands. Nothing was done to strengthen the roof trusses and the new owner threw open the doors of the building as a roller-skating rink.

The original design was passed upon by the city building-inspection department, but no subsequent investigation was made. Owing to the fact that no lives were lost there is little local interest in the failure, and the building-inspection department has never made more than a cursory examination to determine the cause; the results of this examination are not made public. As is perhaps natural in such a case there is an evident desire on the part of all concerned to let the matter rest, and the foregoing facts were obtained only by careful inquiries and painstaking search in the field, which will explain the absence of data which would be of value.

The building is to be reconstructed on approximately the same lines as before, but the roof will be carried on two-hinged timber latticed arches of the same (110-ft.) span. The architects for rebuilding the structure are Scott & Welch, Salt Lake City.

Unit Stresses in the Design of Car Truck Frames

In the evolution of railway rolling stock nearly all the progress has been the result of a cut-and-try process. Parts have been dimensioned according to judgment and experience, and until recent years little attention has been paid to computations of stresses and proportioning of parts in accordance with the computations. One reason for this, of course, is the impossibility of any accurate determination of the stresses that result from dynamic action and shock. It is now generally recognized, however, that approximate computations of stresses are of great assistance in the intelligent design of rolling-stock parts and still give ample room for good judgment in the adoption of final proportions.

Interesting figures have just been made public concerning the unit stresses adopted in the design of the "Fox" type of pressed-steel pedestal trucks, manufactured by the Pressed Steel Car. Co., of Pittsburgh, Penn. While this type of truck has been in extensive use in freight-car service for some 20 years, material changes in the design have been made during that time, not only in general dimensions to correspond with the increased weight of rolling stock, but in the details of various parts to enable them better to withstand shock and distortion.

Each side frame of these trucks is designed to carry a load placed at its center equal to the full M.C.B. rated carrying capacity of both axles of the truck. Under this load the fiber stresses in the truck frame must not exceed half the elastic limit of the metal. In other words, the side frame is designed for double the static loading permissible under the Master Car Builders' rules. This excess of strength is provided to resist vibration, shock, lateral thrust on curves, and other stresses that may occur in excess of the static load. The truck transoms are likewise designed so that the maximum static load which may be imposed upon them will produce a fiber stress not exceeding one-fourth the elastic limit of the metal.

Some Significant Water-Works Census Averages

By Charles W. Sherman*

Engineering News of May 18, 1916, called attention to the appearance of "General Statistics of Cities: 1915," from the United States Bureau of the Census. Certain average statistics relating to water-works, computed from the report, are of sufficient general significance to warrant their presentation separately. Group I includes two cities having populations in excess of 500,000; Group II, 10 cities, 300,000 to 500,000; Group III, 9 cities, 100,000 to 300,000; Group IV, 12 cities, 50,000 to 100,000; and Group V, 35 cities, 30,000 to 50,000 population.

PER CAPITA WATER-WORKS AVERAGES, BY POPULATION GROUPS

	I	II	III	IV	V	Aver.
Capacity of impounding reservoirs, gal.	19,000	46,000	21,000	49,000	19,000	23,000
Capacity of distributing reservoirs and standpipes, gal.	420	500	390	310	520	420
No. of days' supply	2.8	3.3	3.0	2.5	4.1	3.0
Distribution System—Main Pipes and Hydrants:						
Persons per mile of pipe	1,020	610	530	530	390	690
Hydrants per mile of pipe	11.3	8.1	8.3	7.4	6.8	8.9
Hydrants per 1,000 persons	11.0	13.2	15.6	13.9	17.2	13.0
Value of water-works (estimated)†	$45	$45	$39	$34	$38	$42

† These figures would be somewhat increased if the value of the water-supply works of the Metropolitan District of Massachusetts were included.

*With Metcalf & Eddy, Consulting Engineers, 14 Beacon St., Boston, Mass.

Field and Office

Tremie Plant for Concreting Subaqueous Pedestals*

By Kirby Smith†

In the construction of an extension to the boathouse at the United States Naval Training Station, Newport, R. I., a special concreting plant was installed to place the under-water piers or pedestals upon which the columns of the buildings foot. The project provided for 12 concrete and granite piers, ranging in overall height from 8 to 14 ft., the granite extending from El. —0.5 to El. +4.92. This type of pier was proposed because of the excessive ice action at the location of the boathouse, due to the shallow water and strong currents through the boathouse channel. A typical pier is shown in Fig. 1.

Several methods were considered for the placing of the concrete, including (1) premolding, (2) the use of a coffer-dam, and (3) tremie. The first method was rejected as impracticable, because of the uncertainty of the bottom elevation, making it impossible to determine the exact size of blocks, and because of the difficulty of placing and aligning accurately after casting; the second was given up because of the excessive cost and the lack of such apparatus as pumps, a piledriver that would handle sheeting, etc.; while the third—pouring in place by tremie—was decided upon as being the only feasible method, considering the nature of the work, the location, the cost and available plant.

The excavation was made with a ½-yd. bucket mounted on a small power derrick. The material excavated was soft silt and gravel. As the excavation for each pier was completed, the form, having been previously built, was placed. All forms were carefully made of 2x6-in. tongued-and-grooved yellow pine with 4x4-in. rough corner braces. The lower ends of the corner pieces extended about 6 in. below the form proper. After placing, dredged material was backfilled around the lower part of the form to prevent the concrete from working out and to keep the form from moving laterally while pouring. Eight 90-lb. lead mine weights were hung on the outside of each form to overcome buoyancy.

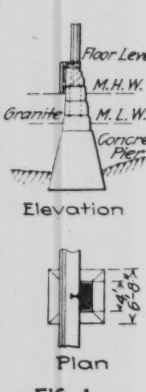


FIG. 1

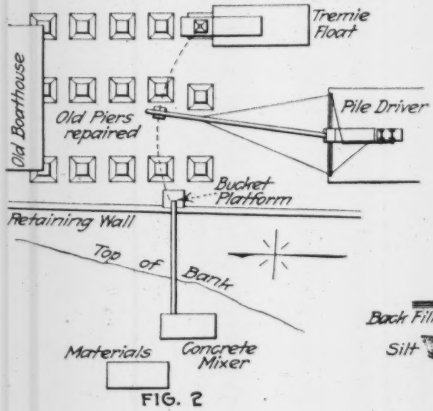


FIG. 2

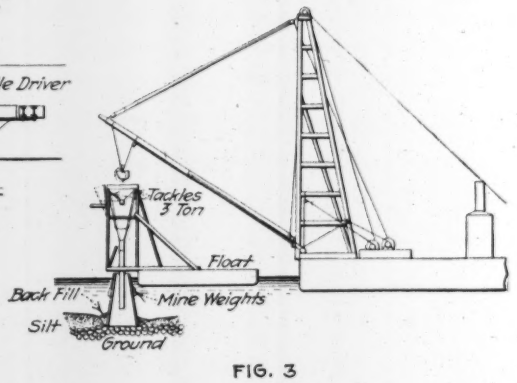


FIG. 3

The layout of the plant (Fig. 2) consisted of a ½-yd. Austin concrete mixer operated by a 10-hp. gasoline motor, a 15x30-ft. steel float carrying the fixed tremie hopper and frame, and a large floating piledriver on which was rigged a jury boom carrying a 9-cu.ft. concrete bucket. A 15-in. tremie pipe was used, suspended from the underside of the fixed hopper by two 3-ton differential hand-operated tackles, capable of raising the tremie easily and rapidly when full of concrete. The tremie was in two sections, one piece 15 ft. long and one piece 2 ft. 6 in., the smaller section being used with the 15-ft. piece to secure sufficient length for the deepest forms. To the upper part of the tremie pipe was attached a hopper 3 ft. high, 36 in. in diameter at the top. The upper, or fixed, hopper had a capacity in excess of the volume of the larger section of pipe. This capacity was necessary in order to insure filling the tremie above the height of the water with the first batch of concrete.

A 1:2:3½ concrete was used. The stone was crushed granite 1 in. and down; sand ¼ in. and under, clean and very well graded. The resulting mix flowed and handled well and remained plastic for a considerable length of time. The concrete was conveyed from the mixer to the bucket, on the lower platform, through a tin-lined wooden chute 12 in. wide and 10 in. deep, the flow being regulated at the mixer. To prevent loss of concrete, however, the lower end of the chute was fitted with a sliding gate and the sides increased in height to 24 in.

The concrete was carried from the loading platform to the large upper hopper by the improvised-derrick boom on the piledriver (Fig. 3). The flow from this hopper into the tremie was regulated by a flat slide valve operated from the platform at the top of the tremie frame. When the large hopper, holding about 25 cu.ft., was filled with concrete, the tremie was lowered to the bottom, then raised so as to be just clear. A sack filled with hay was then placed in the tremie, floating at the surface of the water. The valve in the upper hopper was then quickly

*Civil Engineer, United States Navy, Newport, R. I.

†From Bulletin No. 22, Public Works of the Navy, March, 1916.

opened wide; the flow of concrete forced the bag to the bottom of the pipe, carrying all water out with it. When the tremie was filled to a point about 2 ft. above the outside water surface, it was slacked back so as to rest on the bottom, checking further flow. As more concrete was deposited in the upper hopper and allowed to flow into the tremie pipe, the tremie was slightly raised, permitting the concrete to flow out into the form. After the form had started to fill, the bottom of the pipe was kept submerged in the concrete about 12 in. In this way one surface only was presented to the action of the water. Great care had to be exercised in regulating the height of the tremie, especially when concrete was discharged from the upper hopper, as, with the release of weight of this concrete from the smaller scow, the scow would rise quickly, carrying the tremie with it. Fortunately, only once during the entire work was the concrete in the pipe lost.

The pouring of two piers a day was considered good progress, as the work had to be conducted only at favorable stages of the tide. The form was filled to about mean low water and then later cut off to the required elevation. This insured a firm, hard base for the granite.

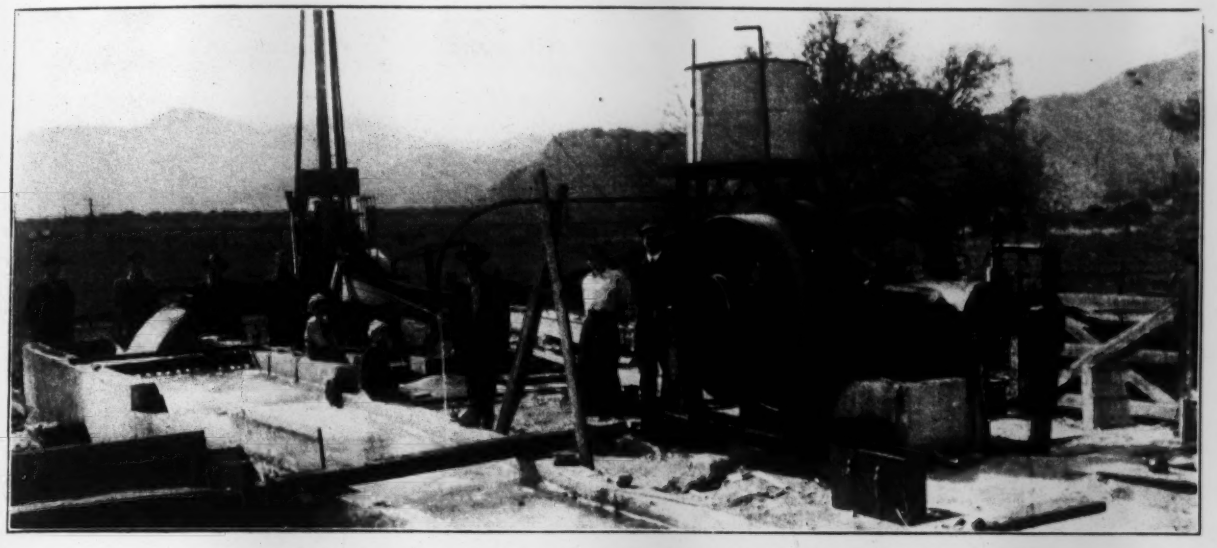
The granite was placed from a small wooden float on which a hand derrick was rigged. The cost of the work, including the cost of rigging the piledriver, building frame for tremie, and cost of the tremie, follows:

	Quantity	Unit Cost	Total Cost Material	Total Cost Labor	Total
Excavation	150 cu.yd.	$1.65 per cu.yd.	.....	.....	$253.52
Forms.....	1,440 sq.ft. contact	0.147 per sq.ft. contact surface; $1.93 per cu.yd. concrete			
	3,640 ft. b.m.	0.058 per ft. b.m.	$104.80	$106.56	211.36
Concrete...	109 cu.yd.	9.91 per cu.yd.	503.90	577.35	1,081.25
Granite....	25 cu.yd.	23.86 per cu.yd.	422.10	174.50	596.68
					$2,142.81

Pumping Plant for Small Irrigation Projects

A type of pumping plant which is becoming extensively adopted in the Southwest is shown in the accompanying illustration. Plants like this are used both for small public water-supplies and on irrigation projects. The use of these small isolated plants for pumping ground waters makes possible the reclamation of many thousands of acres of land in California, Arizona, New Mexico and western Texas, which are not and probably never could be included in projects larger than a few hundred acres.

The plant illustrated is typical of the most recent development in this branch of irrigation engineering. It



IRRIGATION PUMPING PLANT OF GUILD FARMS CO. NEAR TUCSON, ARIZ.

was completed in January for the Guild Farms Co. for the irrigation of about 380 acres of alfalfa and grain lands situated 20 mi. northwest of Tucson, Ariz., in the Santa Cruz valley. The pumping machinery, engine, etc., cost about $1,000 and the well, including drilling and casing with hard-steel stove pipe, 12-gage, cost $7.50 per ft. for the 24-in. section and $5.25 per ft. for the 16-in. section. The total depth of the well is 170 ft., of which 70 ft. is 24 in. and below that 16 in. The cost of the water-supply, including everything, was therefore about $5,000.

The normal level of the water in the well is 53 ft. below the surface. The pump is a two-stage 24-in. centrifugal running at 980 r.p.m. on a vertical shaft in the well, 70 ft. below the surface of the ground. These pumps are made by the Layne & Bowler Co., Houston, Tex. The plant described was installed by the Schweitzer Machine Co., of Tucson and Phœnix, Ariz.

The power is a 60-hp. 4-cycle oil engine made by the Western Gas Engine Co., Los Angeles, Calif., of which the fuel consumption is 6 gal. per hr. (38° Baumé). The well shown was tested by continuous pumping at the rate of 1,900 gal. per min. for 8 days, during which the water was drawn down 17 ft., making the average lift 61 ft. The length of the suction pipe was made 35 ft. The water is pumped into a concrete flume having a permanent weir. Water for domestic purposes (stored in the 1,000-gal. tank shown in the background) and for cooling the engine cylinder is pumped from the flume by means of a 2½-in. centrifugal pump. The pump and engine were subsequently housed at an additional cost of about $200.

It has been found that success in this class of irrigation requires the services of experienced engineers, and the farmers and ranchers are coming to all this. The preliminary work requires a thorough knowledge of the

groundwater conditions of the locality; and the selection and installing of the proper equipment have become a recognized local branch of engineering.

Occasionally groups of pumping plants like the one described are operated by electric motors from a central station. The Tucson Farms Co. operate about 80 pumping plants from the central power station of the Tucson Gas, Electric Light and Power Co. The motor is usually mounted on the same vertical shaft which carries the pump.

Cutting Ground Wire Stops Telephone Trouble

Interference between a grounded telephone line and a power-distribution line in Wisconsin has recently been cured by sectionalizing the overhead ground wire installed on the power poles for lightning protection. This was the case of the Bangor Telephone Co. against the Chicago, Milwaukee & St. Paul Ry., which maintained a 4,400-volt signal supply line.

There was a continuous overhead ground wire about 2 ft. above the power conductors and grounded at each tenth pole. The telephone lines were a half mile away from the power line except in the village. It appeared that the ground wire had become a part of the ground circuit between subscribers near to and far from the exchange. The ground wire was cut for test by the engineering staff of the Wisconsin Railroad Commission and this reduced the interference to such an extent that the railroad company decided to install section-break strain insulators, at the same time removing the ground connections nearest the exchange.

The engineers of the commission believe that many similar situations all over the country could be similarly relieved, to remove the deadlock between the power concerns, which claim the grounded telephone systems to be obsolete, and the small telephone companies which maintain that their service was all right before the power lines began to operate.

Small Dump-Cars Operate on Concreting Trains

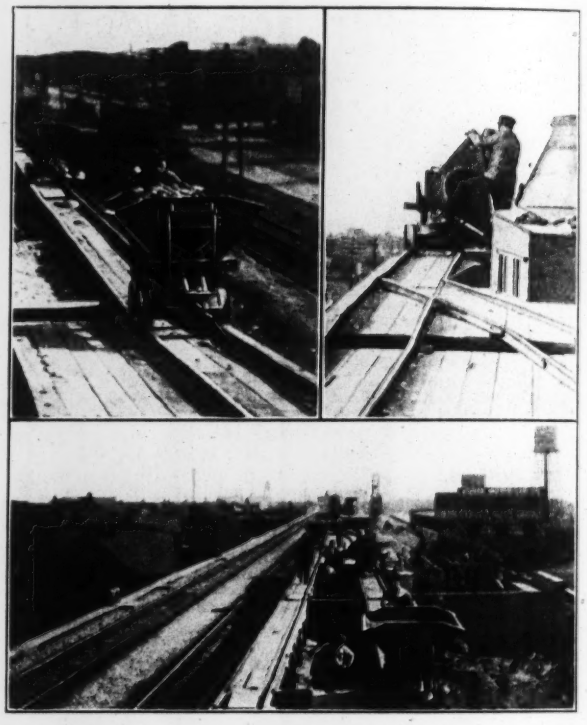
A new method of handling material on concreting trains has been developed on the Chicago track-elevation work of the Chicago, Rock Island & Pacific Ry. In general practice the material cars have temporary run ways for wheelbarrows, by which the materials are delivered to the mixer car.

The first improvement for the Rock Island trains was to equip the gondola cars with permanent runways. These cars are operated only in construction service and run between the Rockdale gravel pit and the Chicago work. As they are loaded directly from the gravel-washing plant, the runways do not interfere with the load. The purpose was to save the time consumed in placing and removing the runways every time a new supply of material cars was attached to the train. It was found also that the men felt safer and that this was reflected in the amount of work done. The use of these permanent runways reduced the cost about 15c. per yd. for labor.

A further step has been taken this year by putting a narrow-gage industrial track on each runway and operating small Koppel V-shaped steel dump-cars. There is a runway on each side of the train, the tracks being connected by a switch on the mixer car so that all the dump-cars run to the same side of the charging hopper. The cars are pushed by hand, but a cable is used to haul the loaded cars up the incline from the top of the gondola cars to the roof of the mixer car.

In the accompanying illustrations the large view shows the train of material cars, with runway and track along each side, and the mixer car at the end. The concrete is being placed in the abutment wall of a subway. One of the upper views shows a loaded car with cable attached for hauling up to the charging hopper. The other view shows the track connections and a car being dumped at the hopper.

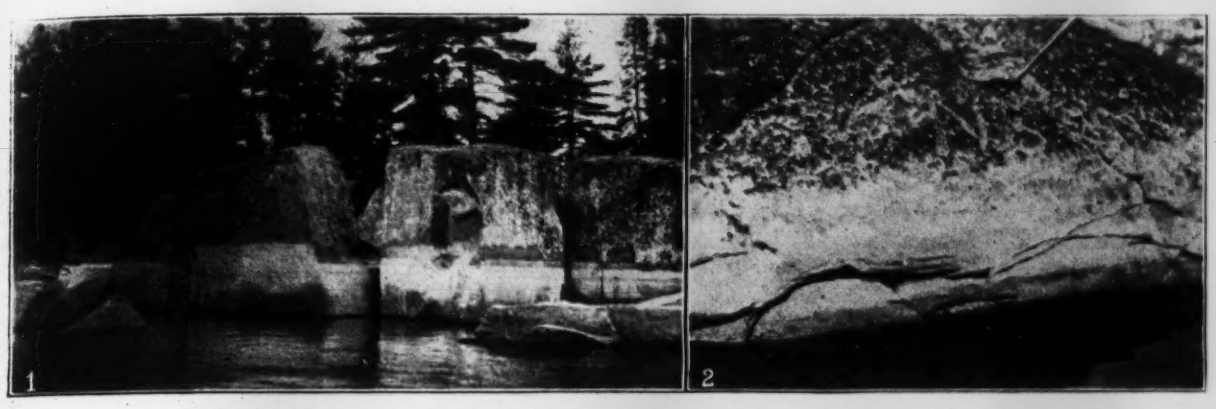
With this equipment and method of working, all wheelbarrows and half the men are eliminated, while a further saving is made of about 20% in the labor cost of handling concrete. This work is being done by company force, and the improved method of handling material was devised by Robert H. Ford, Engineer of Track Elevation.



SMALL DUMP-CARS HANDLING MATERIAL ON CONCRETING TRAINS

Age of Lake-Shore Water Marks

This is the subject of an interesting chapter of the preliminary report of Adolph F. Meyer of St. Paul and Arthur B. White of Toronto, consulting engineers to the International Joint Commission, investigating the water level of the Lake-of-the-Woods. It was necessary to determine the level of the lake before this level was disturbed by the works of man. The report goes into considerable detail as to the significance of the various forms of vegetation as evidence of the former lake stages.



WATER STAINS OR MARKS ON ROCKS ALONG SHORE, LAKE-OF-THE-WOODS
1—Clearly defined marks. 2—Illustrating time required for growths to efface mark

Moss and lichens, common vegetation in northern latitudes, played an important part, the water stains on rocks along the shore being due to the absence or partial absence of one or more forms of these lichens.

It was found that removal of these lichens and the formation of a water mark by standing water might be effected in a comparatively short time, possibly a season, but a number of years were required to substantially change the sharpness of a water line once formed.

These stains, or water marks, are shown in Fig. 1, the marks being higher on the exposed face than around the corner where protected from wave action. The length of time required to alter a mark is illustrated in Fig. 2, where the high-water mark was cut in the rock in 1895.

Fishing for 60-Ft. Girders Washed Out in Flood

By Ernest D. Clabaugh*

A January flood in Southern California played havoc with a bridge of the Atchison, Topeka & Santa Fe Ry., over the Santa Ana, between Richfield and Olive. On the north side 300 ft. of embankment was washed out, on the south side 400 ft. Ten bents were taken out and numerous scattered piles of the trestle, as were also four 60-ft. steel-girder spans and four concrete piers. Three of the spans have been located, but the fourth has entirely disappeared. Two were sticking above the sand, as illustrated in Figs. 1 and 2, while the third was discovered by kicking it while wading in the river.

In repairing the bridge a temporary pile trestle was built out from where the embankment had been and along the former site of the girders. This trestle consisted of five-pile bents spaced 14 ft. c. to c. and of two 4-ply chords. The piles were driven to refusal, the average penetration being 28 ft. The projection above ground was 8 ft., giving a pile length below cutoff of 36 ft.

During piledriving two of the concrete piers were encountered. These were found to be at depths of 14 and 17 ft. below the surface of the sand. The piers were definitely located by means of a water jet and were found to be turned over on their sides, having fallen slightly upstream. One pier had been left standing by the flood, and resting against this and in line with the track was one of the spans with one end sticking down into the sand a distance of 28 ft. (Fig. 2).

*Bridge Inspector, Atchison, Topeka & Santa Fe Ry., 115 So. Olive St., Anaheim, Calif.

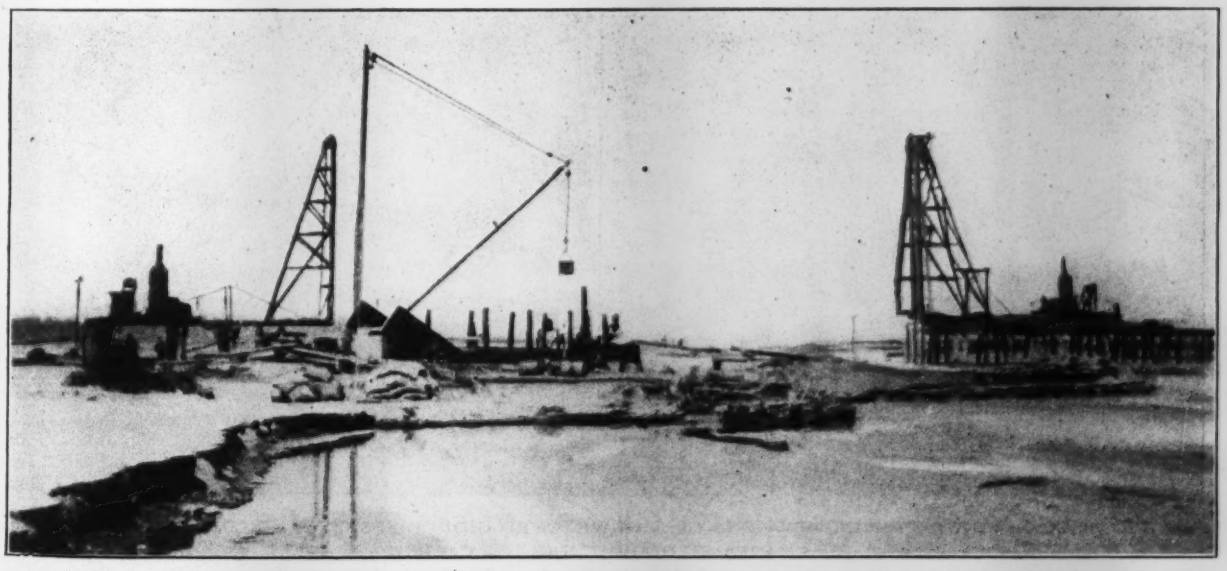


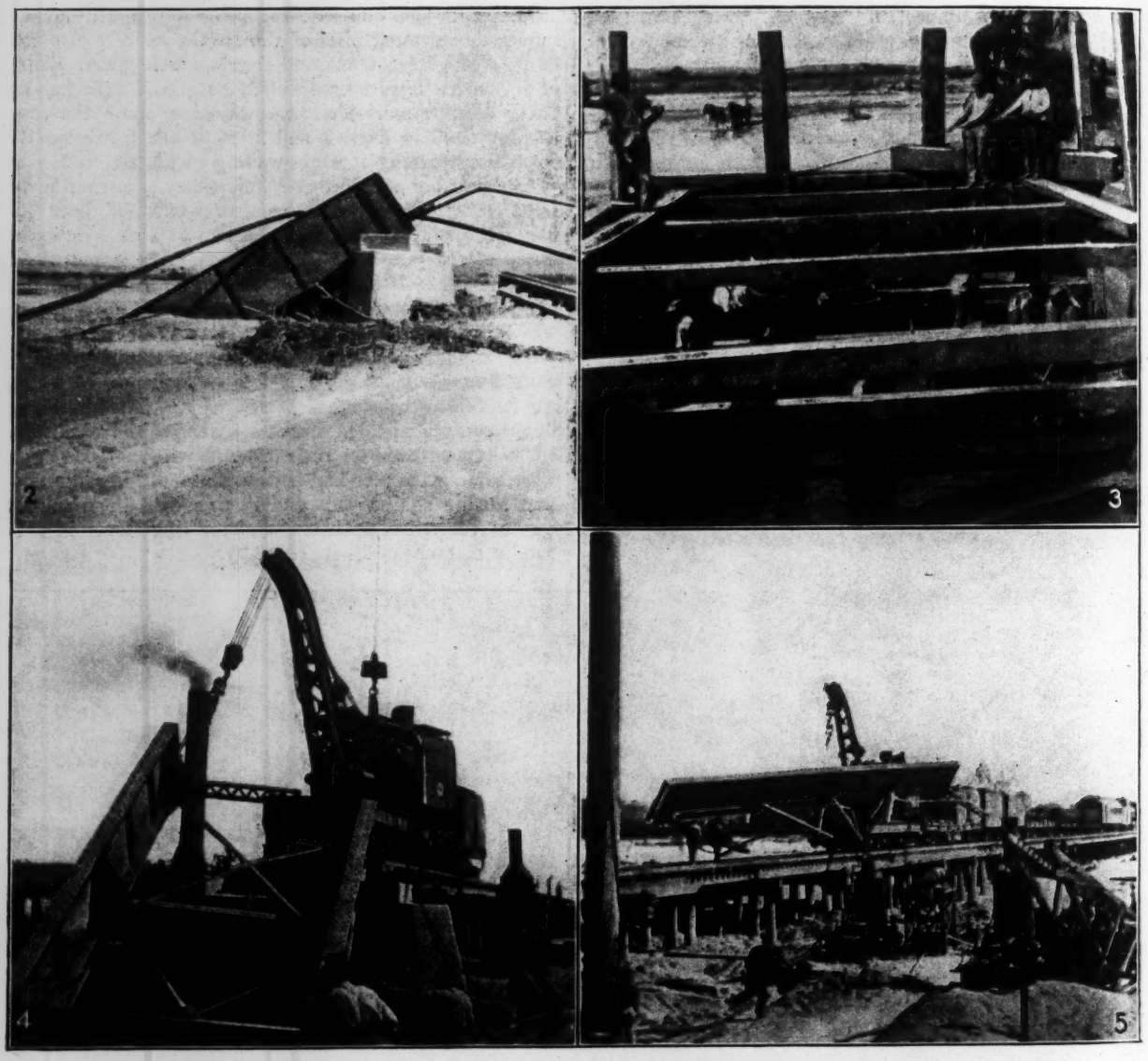
FIG. 1. EXCAVATING FOR GIRDER SPAN; PILEDRIVERS WORKING AT BOTH ENDS OF TRESTLE

This span had to be removed in order to extend the trestle, and this operation was accomplished by sinking a timber coffer-dam inclosing the girder. The sand was excavated from inside by a time-consuming method made necessary by the accumulation of débris and bridge ties that had collected about the girder, making it impossible to use a clamshell bucket. All the dirt had to be shoveled into boxes, which were raised by a derrick (Fig. 1) and dumped to one side of the coffer-dam.

Water in the river was diverted from around the girders by means of a sandbag levee. To remove the water from the inside of the coffer-dam two 8-in. centrifugal pumps had to work continuously. At 20-ft. depth these pumps could barely keep up with the inflowing water. At this depth a 50-ton wrecking train pulled the girder the remaining distance out of the sand. As long as the span was intact the wrecker could only shake it. The sides were parted by an oxyhydrogen flame; then by alternately raising and lowering the ends of each girder, the two were readily removed, one at a time. As soon as the span was out of the way the gap was closed and traffic resumed.

The other spans will be exhumed as soon as the water in the river lowers sufficiently to make pumping practicable. These spans lie almost parallel to and 3 or 4 ft. below the surface of the ground and their removal will not present unusual difficulty.

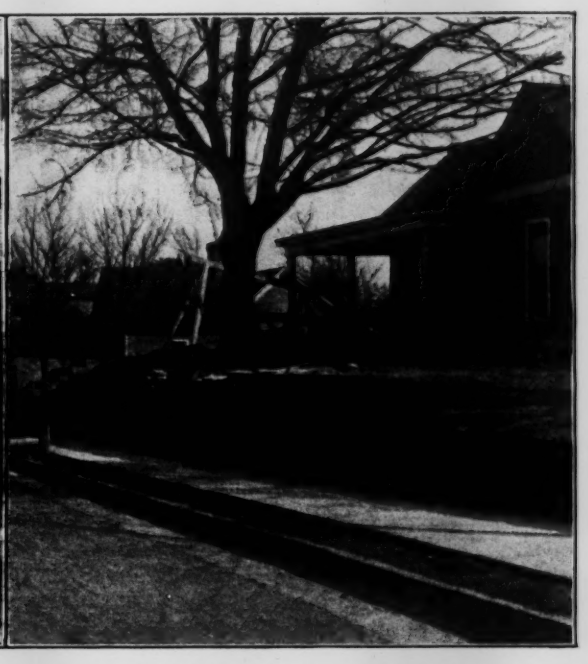
This washout stopped traffic on this line from the latter part of January until the end of April, but this did not seriously interfere with the business of the railway as the line was used primarily as a cut-off for freight traffic between San Bernardino and San Diego and this business was readily diverted through Fullerton, the haul being lengthened by about 10 mi.



FIGS. 2 TO 5. VIEWS SHOWING HOW ONE OF THE WRECKED GIRDER SPANS WAS RECLAIMED ON THE SANTA FE BRIDGE IN OLIVE, CALIF.

Fig. 2—Steel girder spans 60 ft. long, almost buried in sand. Fig. 3—Coffer-dam inclosing girder span. Fig. 4—Recovering span, using wrecking train. Fig. 5—Taking girder ashore on wrecking train

LOWERING SLABS OF FORMER SIDEWALK



SIDEWALK RELAID AT NEW GRADE

Lowering Concrete Sidewalk to New Grade

Last summer, after a number of streets in Dyersburg, Tenn., had been improved, it was found that the concrete sidewalks would either have to be rebuilt or lowered to new grade. In view of their excellent condition it was decided to lower them. The following costs are for a typical section of this work:

Lineal feet of sidewalk	74 ft.
Width of sidewalk	5 ft.
Cost of labor	$26.40
Cement: 8 sacks at 60c	4.80
Sand: ¼ yd. at $3	.75
Sand: ¾ yd. at $1.40	1.05
Total cost	$33.00

Excavation on this section amounted to 18 cu.yd. and the cost of lowering to 8.8c. per sq.ft. The sidewalk in Dyersburg is laid by contract at 12½c. per sq.ft.—1:2 (limestone screenings) for ¾-in. top; 1:3:5 for 3¼-in. base. The saving accomplished by lowering instead of rebuilding therefore amounted to 3.7c. per sq.ft. The contractor has agreed to lower sidewalks in the future for 8c. per sq.ft.

Bridge Piers in Sumatra

By J. W. Swaren*

The geological formation of the island of Sumatra is volcanic and its streams have no gravel. There is, however, plenty of sand. As a usual thing, the roads are little more than trails, with only the deeper streams crossed by bridges. Recently, a hydroelectric plant was installed in the interior, and it was necessary to build a road wide enough to accommodate bullock trucks, and bridges of sufficient strength to bear up the heaviest parts of Pelton wheels. Timber stringers set on distinctly Sumatran piers were used. The cement for these piers was transported in sacks carried on the heads of the natives.

*Hayward, Calif.

Arriving at the site, an approximate 1:4 mixture of dry cement and dry river sand was hand mixed. The mixture was then shoveled dry into empty burlap bags, and these were piled sandbag fashion in the stream, as shown in the photograph. The first tropical rain that came along was supposed to complete the concrete.

In the far interior, some of the natives looked on these bridges as traps set by the white men and still clung

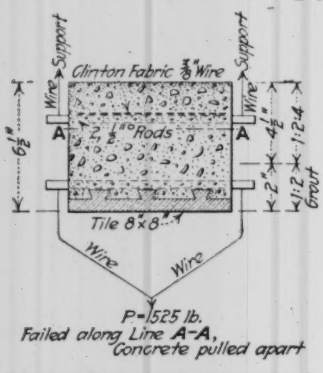
SUMATRAN BRIDGE PIERS BUILT OF BAGS OF 1:4 CEMENT AND SAND

to the use of their fords. However, the more daring spirits mustered up sufficient courage to cross these diabolical contrivances, and their example was soon followed by the more timid.

Test of Adhesion of Face Tile to Concrete Backing

Vitrified clay tile are used as smoke and blast protection for the concrete encasement of the Boston & Albany R.R. crossing over the New York, New Haven & Hartford R.R. tracks at Worcester, Mass. A test was made

to determine the gripping effect between tile and concrete. Very satisfactory adhesion strength was found; in fact the strength of the grip exceeded the strength of 9-day-old concrete. The following particulars are given by F. B. Freeman, Chief Engineer of the Boston & Albany R.R. The top of the locomotive stacks comes within 3 ft. of the under surface of the bridge. The tile selected for these conditions was burned specially hard, almost to vitrification. To prevent excessive warping in the burning, the size was made small, 8x8 in. The tile is 1 in. thick, and has on the back three longitudinal dovetail ribs 1⅜ in. wide on top, 1⅛ in. wide at the neck and ½ in. high.



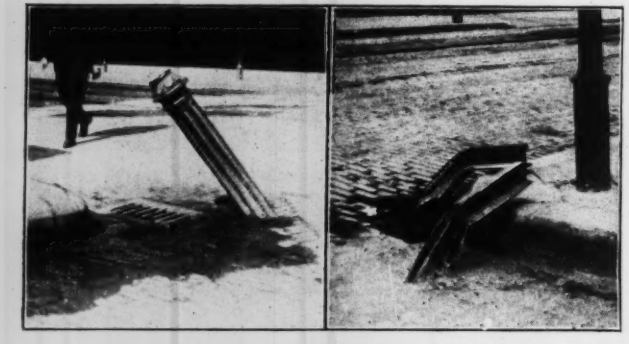
TEST OF BLAST-PROTECTION TILE FOR BOND TO CONCRETE

The test block of concrete was made 6½ in. high, with 1:2 mortar in the lower inch (2 in. from face of tile), and 1:2:4 concrete for the 4½ in. above this. After 9 days' hardening the test was made in the way shown by the sketch. The average of the strengths obtained in several tests was 1,525 lb.

The rods of which load was applied were molded in the specimen, and probably had a certain amount of support from the concrete itself. However, if a well-marked plane of weakness existed at contact of tile and concrete, embedment of the rods would not delay the splitting away of the tile very long.

NOTES

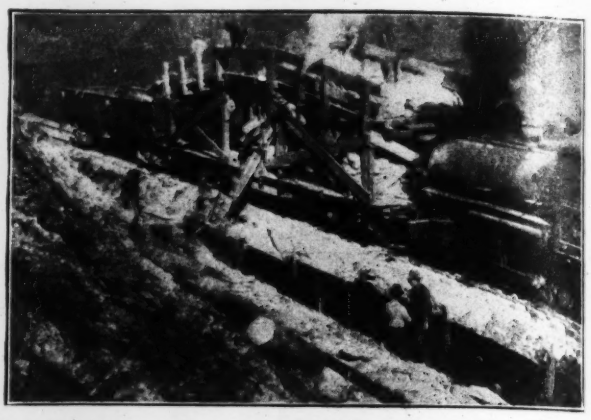
Catchbasin Inlet Protectors are shown in the accompanying illustrations. They are examples of the practice of the Galveston, Tex., street commissioner. The sidewalk curbs are practically sharp cornered (not over 2-ft. radius); and the catchbasin inlet is either in the street, as the first view shows,



CATCHBASIN INLET PROTECTORS, GALVESTON, TEX.

or at the point of intersection, as in the second view. The street authorities are evidently inclined to protect the inlets and curb corners at any cost. Not all the curb corners in the business section of Galveston are thus "armored."

A Concreting Train for Building a Retaining Wall was devised by G. C. Earl, Assistant Division Engineer of the Bingham & Garfield R.R. This railway winds around and among the Utah Copper Co.'s immense mines at Bingham, Utah, and



CONCRETING TRAIN; UTAH COPPER CO., BINGHAM, UTAH

is used primarily to bring ore from the various benches or berms (where it is excavated by steam shovel) to the concentrating plant at Garfield. The railway climbs the mountain sides on 4% grades by means of switchbacks, so that the tracks are often but a short distance apart horizontally, separated by steep slopes. Between two such laps, or levels of track, it was recently necessary to construct a retaining wall, and the view shown illustrates the construction method and plant. On account of the track above (not shown) a permanent plant could not be placed on the uphill side, so a movable plant was designed to work from one of the main tracks of the lower level. When necessary, this outfit can be moved to a siding to permit train operation. The plant consists of a locomotive, a flat-car with an overhead bridge, a 10-cu.ft. concrete mixer and a gondola car for gravel. A half day's supply of cement is kept on the mixer car. A track and a 4-wheeled skip bring the gravel from the gondola to the mixer hopper. The track is laid on the bridge incline and on top of the gravel in the gondola, until it is necessary to support it on horses as the car is emptied. To bring the skip up the incline, a hoist is provided below, alongside the mixer. Both mixer and hoist are operated by steam from the locomotive. The mixer is charged from the overhead bridge and discharges into a chute which carries the concrete to the forms. The bridge car is reversible and can be coupled to either end of the gondola. The capacity of the plant averages 125 yd. per day.

Costs to Build Spur Tracks are given in a recent issue of "Power." A 778-ft. spur, including a 1,660-ft. trestle, cost $6,121. Spurs built entirely on the surface cost from $1.30 to $2.40 per mi. Following are the items for spurs of different lengths. The work was in Massachusetts:

SPUR TRACK; LENGTH, 562 FT.

Ballast	73 cu.yd.	@	$0.75	$55
Ties	281	@	.52	146
Rail, 56-lb. T	9.32 tons	@	28.54	266
Joints	24	@	.62	15
Split switch and frog				75
Spikes	3.3 kegs	@	4.10	13
Tracklaying	562 ft.	@	.25	140
Teaming on track material	12 tons	@	1.25	15
Total				$725

SPUR TRACK; LENGTH, 693 FT.

Grading, earth	682 cu.yd.	@	$0.30	$205
Ballast, cinder	234 cu.yd.	@	.25	58
Ties	304	@	.51	155
Rail, 60-lb. T	10.79 tons	@	28.54	308
Joints	60	@	.65	39
Spikes	4 kegs	@	4.10	16
Switches and frogs				202
Tracklaying	693 ft.	@	.25	173
Teaming on track material	17 tons	@	1.25	21
Total				$1,177

TRESTLE AND SPUR; LENGTH, 778 FT.
(Construction: Timber on Concrete Piers. Trestle About 160 Ft. Long)

Excavation	202 cu.yd.	@	$0.50	$101
Concrete in piers and abutments	208.5 cu.yd.	@	5.75	1,199
Timber in trestle, including ties on trestle,	40 M bd. ft.	@	60.00	2,460
Ellis bumper (1)				70
Ladder (1)				4
Earth-filling	4,200 cu.yd.	@	.31	1,302
Ties	314	@	.59	185
Rails, 75-lb. T (laying)	778 ft. track	@	.25	194
Rails, 75-lb. T	17.36 tons	@	30.63	532
Joints, continuous	52	@	1.64	85
Spikes	4.1 kegs	@	4.10	17
Teaming on track material	26 tons	@	1.25	32
Total				$6,121

Editorials

Safety for Workmen's Eyes

The widespread "Safety First" movement, which has had such a wonderful effect in the railway service and in industrial establishments in reducing the number of accidents to workmen, is receiving attention also in engineering-construction work. It is much more difficult to train the shifting force of workmen employed on the average construction contract to observe safety rules than it is to educate the more permanent employees who work on a railway or in a machine shop. The engineer who wishes to reduce accidents to those working under him, therefore, usually gives chief attention to guarding machinery and to safe handling of explosives.

It seems worth while to call attention to one simple safety appliance that ought to be much more generally used on construction work, and that is goggles for the protection of the eyes where men are engaged in chipping stone or metal or in other work where the eyes are likely to be struck by flying particles. In a special bulletin on "Industrial Accident Prevention," issued by the New York Department of Labor, several pages are devoted to protection of the eyes. It is fortunate that the wide use of automobile goggles makes it possible to obtain for workmen almost anywhere now, at a trifling sum, eye-protecting apparatus that is generally entirely satisfactory. Not only this, but the custom of wearing goggles, which the automobile has established, has gone a long way toward eliminating the prejudice against their use and made it much easier to induce workmen to wear them.

Creeping and Heaving of Concrete-Road Slabs

Sufficient experience has accumulated in concrete-road construction so that most concrete roads as now built have a hard and durable surface capable of resisting the wear of traffic. Difficulties are being experienced, however, especially in northern latitudes, with the rising up of one slab above another at transverse joints and also with the gradual widening of longitudinal cracks.

In concrete roads built south of the latitude of New York little trouble seems to be experienced with cracking. The generally adopted plan of filling a crack with melted tar and covering it with sand appears to make the road as good as new, involves little expense and appears to be unobjectionable, except on the possible ground of the appearance of the pavement.

In roads located farther north, however, there are cases where longitudinal cracks in a concrete road have given much more trouble. In certain cases the crack gradually widens until sometimes the two sides are separated as much as two or three inches. Trouble has been experienced occasionally also with the slabs on the two sides of a longitudinal crack getting out of level. A concrete road in North Andover, Mass., was so heaved by the frost last winter that the concrete on one side of a crack through the center was 2 in. higher than on the other. When the frost came out of the ground, the slabs settled back to place; but the cracks which were originally hair cracks were widened to about ½ in. in some places. The cause of this movement on this particular road, and probably on most roads where trouble has been experienced, is poor drainage of the subgrade. In any climate where the frost penetrates deeply, if the subgrade is saturated with water, there is bound to be considerable upward movement as the ground freezes; and if one side of the road is wetter than the other, the heaving will be unequal.

Every road engineer of long experience knows that in constructing all of the older kinds of highways, from dirt roads to macadam, good drainage is the first essential to success. Possibly some engineers may have had the idea that, because a concrete road forms a tight roof over the subgrade and distributes the load equally over it, thorough drainage is less necessary. The experience previously mentioned indicates that it is well worth while to take pains with drainage where a concrete road is to be built.

There are, however, engineers who are fearful of the results from longitudinal cracking in a concrete road even with the drainage thoroughly taken care of. It was doubtless this feeling on the part of many engineers that led to the recommendation by the Concrete Road Conference, last February, that on concrete roads 20 ft. or more in width reinforcement should be embedded to prevent longitudinal cracking. There is controversy, however, as to how much reinforcement is necessary to make a road safe against cracks and as to whether the reinforcement will not be broken if a crack does occur, so that it will not prevent the gradual separation of the slabs.

Where such separation happens, it is evident that the slabs creep on the subgrade. Such creeping has been known to take place even when the subgrade was made entirely flat. Probably the water percolating into and freezing in the central crack is sufficient to account for this movement.

A plan has been suggested and is under consideration for adoption on an important road some five miles in length in the vicinity of New York City to obviate the risk of trouble from the creeping of slabs and thoroughly bond them to the subgrade. The road in question runs through a traprock district where good material for a telford base is obtainable at nominal cost. It is proposed after completing and rolling the subgrade to place upon it a layer of telford stone, exactly as would be done for building an ordinary macadam road. This telford will be filled with spalls and compacted with a steam roller. The telford stone will in this way be thoroughly bonded with the subgrade. It is proposed to fill the voids in this layer of telford with portland-cement grout and then lay on top of it, before the grout begins to set, a layer of concrete 3 to 4 in. thick. If made in this way,

the concrete top should be perfectly monolithic with the telford stone underneath. The whole, in fact, would form a layer of concrete 10 to 12 in. in thickness. The under surface of this concrete would be so thoroughly bonded to the subgrade that no creeping movement would be possible. Apparently the cost of constructing a road in this way should be little if any more than the cost of constructing the ordinary single-course concrete road. The telford base should be laid and grouted for about 55 to 60c. per sq.yd., and 50c. to 60c. per sq.yd. should be sufficient to cover the cost of the top layer of concrete.

This type of concrete-road construction might have notable advantages where there is available plenty of local stone that is too soft to stand wear. This stone could be used for the telford layer, reducing by one-half the amount of hard stone or gravel necessary to ship in for the finishing concrete. It would have the advantage, moreover, of giving a better bond between the subgrade and the concrete, and between the upper and lower courses, than is apt to be obtained in the ordinary two-course construction.

It is worth consideration, moreover, whether this type of construction would not be especially advantageous where a concrete road is built without transverse joints, as is being practiced now by a number of road engineers. It seems reasonable to believe that with the thorough bonding to the subgrade, which would take place in this construction, the contraction due to temperature or moisture changes would be so distributed over the road that the tendency to cracking might be greatly reduced.

We commend the suggestion to engineers interested in concrete-road construction and shall be pleased to receive any criticisms or suggestions in connection with it.

Do Elevated Railways Need the Block System?

The rear collision on the Manhattan Elevated Ry. on June 8, in which the motorman of a train was killed and a number of passengers were injured, has revived public agitation to have the block system adopted on the elevated lines. There is practically no disagreement from the proposition that the block system, in some form or other, is the only reasonably safe method of operating trains in general railway service. Under the special conditions of elevated railway service, however, the remarkable record of safe train movement under densely crowded traffic, which has been shown on the Manhattan elevated lines for many years, demonstrates beyond question that it is possible to operate such railways with a marvellously high degree of safety without the use of the block system. It is stated that in the nine years since 1907 the elevated railway lines of New York City have carried over 3,000,000,000 passengers of whom only five have been killed in train accidents.

The collision on June 8 would hardly have been prevented had a block system been in force. The motorman who met his death as a result of the accident ran his train in broad daylight into a standing train of empty cars, which he approached on a long stretch of straight track. Since some defect in his mental machinery prevented him from stopping his train with the brake power under his hand, although his error cost him his life, it is fair to conclude that he would have been no more likely to stop had a semaphore arm beside the track stood at danger.

The motorman did this moreover, it is to be noted, in disregard of a rule that the first car of a train must not run alongside a station platform until the last car of the preceding train has passed the opposite end of the platform. At present during the rush hours, the trains on the heaviest traffic lines of the Manhattan elevated are run under as low a headway as 38 sec. It would be possible, of course, to install upon the elevated lines automatic stop devices such as have been put into use on the express lines in the subway. Such automatic stop devices, although they have never been developed to a point where they have commended themselves for use on railway lines of general traffic, are entirely practicable for roads operated under such conditions as the subway or the elevated lines. If such automatic stops were installed on the elevated, however, it would obviously necessitate an increase in the spacing between trains, and corresponding reduction in traffic capacity. With the slow speeds and regular stops in force on the elevated, and the record of safety which past experience has shown, it would be difficult to demonstrate any material gain in safety through the installation of automatic stops.

Safe Railway-Bridge Floors

A railway-bridge floor illustrated and described in our issue of June 8, page 1095, appears to call for some comment in this place. We published this article, as we publish many articles, as a matter of news and information, regardless of whether the design conformed to the opinions of the editors or not.

The particular feature of this floor was the omission of inside guard rails, and according to the statement furnished us, this construction had been used for some years with satisfactory results and derailed wheels had been guided safely across the bridge. Without questioning this statement, we wish here to give the strongest possible emphasis to the statement that on all railway bridges over which passenger trains run at ordinary speed a bridge floor should be furnished with both inside guard rails and outside guard timbers.

More than 25 years ago two terrible railway-bridge disasters—one occurring at White River on the Central Vermont Ry. in 1887, the other at St. George, Ont., on the Grand Trunk Ry. in 1889—called the attention of railway-bridge engineers the country over to the need of better railway-bridge floors and had unquestionably an enormous influence in improving practice.

Most railway engineers know perfectly well the reasons that make the use of both inside guard rails and outside guard timbers advisable. It may be well to point out here, however, for the benefit of some of the younger men in the profession that the inside guard rail tends to keep the wheels of a derailed truck in line by guiding the back of the wheel and will in most cases hold the truck in line and keep the wheel from striking the outside guard timber. Where, however, inside guard rails are omitted, if a pair of derailed wheels swerve from a straight line until they strike the timber guard rail, there is great danger that they will cut into and mount this timber instead of sliding along it. This is particularly apt to be the case, because, when the leading wheel strikes the timber, the resistance is interposed at a point that

tends very strongly to slew the truck farther out of line, whereas the resistance imposed by the inside guard rail tends to bring the truck back into line.

If there were any large expense involved in placing inside guard rails on a bridge floor, those who attempt to economize by omitting them might have some small basis for defense; but every railway company, even those forced to practice the greatest economy, always has worn rails which can be used at a cost no greater than their value for scrap. The best practice, of course, would call for an inside guard rail as heavy as the track rail; but if extreme economy is imperative, a railway may use old rails for its bridge guard rails and escape censure in case of accident. It cannot do this if it omits inside guard rails entirely.

A pertinent illustration of what has been just said is the recent wreck of a passenger train on the Rock Island road in which 13 people were killed, reported on page 1200 of this issue. It is true that the bridge where the accident occurred was in bad shape because of a flood, but the engine, tender and first two cars crossed in safety and the first car derailed was the smoker. One truck of this car appears to have so bunched the ties that the following car went into the river. It is probable that inside guard rails might have kept this truck in line and saved the train.

A One Hundred Million Dollar Debate

No more remarkable meeting has ever been held in the auditorium of the Engineering Societies' Building in New York City, than that which has been daily assembled there from June 1 to June 15. On one side of a long table extending entirely across the stage were seated a dozen or more men, heads of the railway trainmen's unions which have served a demand on the railway companies of the United States for the establishment of the eight-hour day. Seated on the opposite side of this table were a score of railway officials, the authorized representatives of the railway companies. An audience of 500 to 800 men filled the auditorium, representatives of the local lodges of engineers, firemen, conductors, brakemen, etc., listening to the debate in progress on the stage with a close attention that one seldom sees displayed by an audience in this auditorium or any other.

It is safe to say that this discussion, involving as it does nearly half a million employees and an annual outlay estimated at $100,000,000, is the most gigantic example of "collective bargaining" that has ever been seen in the world. It is much to be regretted that after two weeks of discussion and endeavor to reach a common understanding the conference was ended without agreement.

The railways submitted a tentative offer to grant the trainmen's demand for an eight-hour day, provided that in connection with the demand a general investigation and revision of existing rates of wages should be undertaken with a view to modification according to the following principles: (a) No double compensation to be paid for the same time or service; (b) the same classification for purposes of compensation to be applied to all members of a train and engine crew. The trainmen's representatives, however, refused to accept this.

The conference was adjourned to be reassembled again after the railway unions have taken a vote on the proposition which will be laid before them by their leaders. The final proposition of the chairman of the railway representatives, Elisha Lee, was that the matters in controversy should be referred for decision to some disinterested tribunal for investigation and decision, preferably the Interstate Commerce Commission, or else to arbitration in accordance with the provisions of the Newlands act.

A. B. Garretson, representing the Order of Railway Conductors and chairman of the committee representing the unions, in declining this proposal declared that the Interstate Commerce Commission was a rate-fixing body and had no legal authority to fix rates of wages. Concerning arbitration he declared that the sentiment of the employees was against it and that "until a condition has arisen that may threaten the interruption of traffic, I and the men behind me will never arbitrate."

The prospect that this deadlock between the railways and their employees may be carried to the ultimate end of a general strike is so threatening that the Chamber of Commerce of the United States, representing practically all the leading commercial organizations of the country, has urged upon Congress the adoption of a joint resolution instructing the Interstate Commerce Commission to investigate at once the demands of the employees, the position of the railway companies in connection therewith, and the conditions of employment and pay of all classes of railway employees in the United States in comparison with the wages paid in other industries involving similar skill and risk.

How Can Engineering Societies Discipline Members?

A subject which is receiving a good deal of earnest attention at the present time, is the exercise of discipline in the national engineering societies. There is a general demand that the engineering societies shall exert a greater influence in the affairs of the profession than they have in the past. It is said, with reason, that membership in a national engineering society with high professional standards ought to be in itself a certificate of good character, and that if any member becomes a discredit to the profession his connection with the society should be severed.

Every one probably recognizes that an engineer of dishonesty injures not only his own reputation, but to a certain extent that of the entire profession. It is the very essence of professional work that the one undertaking it shall be faithful to the trust imposed upon him. The public will condone to a certain extent ignorance and incompetence for it recognizes that to err is human; but no excuse whatever can be made for the professional man who is false to the trust imposed upon him.

Most engineers will agree, therefore, that the member of an engineering society who is found guilty of dishonesty of any sort should be promptly expelled. Current opinion on the matter is well illustrated by the recent letter ballot in the American Society of Civil Engineers. The question asked of the members was: "The Board of Direction now elects members in all grades; shall it also have power to expel and discipline?" Of the ballots cast about 85% were affirmative.

It should be said that the constitution of the society already gives the Board of Direction power to discipline and expel members but it provides that an appeal may be taken from the action of the Board of Direction to the membership of the society which shall vote on the case by letter ballot.

It is difficult to see any justification for such an appeal. If the governing board of a society will not give enough attention to a case to decide it justly, it is difficult to believe that several thousand members, scattered all over the country, could or would obtain sufficient knowledge of the case to deal with it more wisely and justly than the board itself. Probably the recent ballot will be considered to confer upon the committee which is to revise the society's constitution authority to eliminate the clause providing for an appeal to the whole membership.

The constitution of each of the national societies empowers the governing body of the society to deal with cases of discipline to the extent of expelling a member and by inference to administer any lesser punishment such as a formal or informal reprimand. The practical question now is, to what extent are the governing bodies of these societies competent to take up this work? Of course if the discipline of members is to go no further in the future than it has in the past, that is to say, if cases of the expulsion of members are to be confined to men who have been found guilty of crime, the governing boards of the societies can doubtless continue to handle these cases as they occur. There is, however, a growing demand that the dishonest engineer, even though not criminally convicted, should be disciplined.

To one familiar with the manner in which the business of a national engineering society is carried on, it seems very doubtful whether the governing boards will be able or willing to handle these questions of discipline if they are to assume any considerable proportions. Most of the men elected to membership in the Board of Direction of the American Society of Civil Engineers, or the Council of the American Society of Mechanical Engineers, or similar governing bodies in other societies are busy men of affairs who can spend comparatively little time in attendance upon meetings, especially when, as is often the case, attendance upon meetings may mean travel of a thousand miles or more.

It will be evident that in any case where a member of a society has had charges brought against him sufficiently serious so that his conviction would mean expulsion from the society the elementary principles of justice require that he should have the right to appear in person before the board which is to pass upon his case and to present his defence with the aid of witnesses, if he desires. The constitution of the American Institute of Mining Engineers contains a clause that a member accused may appear in his own defence before the board and examine witnesses.

The governing bodies of the national engineering societies generally hold regular meetings once a month, and the meetings continue usually not more than two or three hours. It will readily be seen that in a single complicated case of discipline where it became necessary to try a member on charges, if the member should make a vigorous defence the case might occupy as much time as the governing board would require for all the rest of its year's work. Suppose half a dozen or a dozen such cases of the discipline of members were to be brought annually before a national society's governing board it is obvious that the board would be swamped and the members would adopt almost any means of shirking the task before them.

If this is a correct view of the case, it would appear that about the only practical manner in which a national society can undertake the discipline of its members on any extended scale would be to delegate the work to a judicial committee. It should be possible within the large membership of any of the national societies to find three men who have reached a time of life and a degree of success where they are no longer bound down to routine work and who have the necessary leisure combined with a willingness to use that leisure for the benefit of their profession. It goes without saying that the men should be of such standing that they would command the respect of the membership and they should be men of judicial mind so that they could deal with any case that came before them without prejudice.

There is another very important reason why discipline in a national society must almost of necessity be delegated to some such small body if it is to be exercised on any large scale. It may be necessary to exercise discipline upon members resident anywhere in the United States and even in foreign countries. Suppose, for example, charges were brought against a member resident in Galveston. It would be manifestly unjust to compel that member to travel all the way to New York City with his witnesses to present his defence; in comparatively few cases would it be possible for an accused member to go to such an expense. Of course, there are many cases in which the charges and the reply could be presented in written form on both sides and the work for the judicial committee would be comparatively easy. There is always a chance, however, that to go to the bottom of a matter it would be necessary to hold a hearing in the locality where the member resides and a judicial committee could do this, if necessary, while it would be impracticable for the twenty or more men composing the Society's governing board.

Three years ago the committee which drafted a code of ethics for the American Society of Mechanical Engineers which was later adopted by that society recognized that sooner or later disagreement would arise concerning the interpretation of that code, and so provision was made for a standing arbitration committee. It was foreseen, however, that if that committee were to hold public hearings for the examination of witnesses on opposite sides in dealing with such questions, the time consumed would make it difficult or impossible to get anyone to serve on such a committee. It was, therefore, provided that the committee should act only to arbitrate differences between any two members on the interpretation of the code, and its action should be based upon a statement in writing submitted by the disagreeing members. This plan, however, has never been carried into effect.

The above presents some of the difficulties in the way of effective discipline in the national engineering societies. It is easy to agree as to the desirability of such discipline, but very difficult to work out a practical method by which it may be exercised. Until some practical method can be devised and put in operation, the governing boards of the societies will doubtless avoid the exercise of discipline wherever they can, as they have in the past.

Letters to the Editor

Designer of Flat-Slab Floor Defends System

Sir—In the article "Extensometer Tests on Three Types of Concrete Floors," *Engineering News*, May 25, p. 992, you have unfortunately reversed the order of importance I assigned to the two causes of high stresses in the wall panels. Nowhere have I stated that the high stresses "were due mostly to the testing of the slab before it has reached its final set." In the discussion from which you abstracted your article, I referred to the misplacement of steel as "the most far-reaching one of the two conditions that obtained over the wall panels at the time of the test" and to the uncured state of the concrete as "the lesser one of two conditions that affected the wall panels."

In your editorial discussion you appear to lean toward the reasonableness of the contention that the concrete had not attained its full strength, although there is a decided difference of opinion among well informed engineers as to the extent the condition of the concrete may have affected the test, and you dismiss the incontrovertible fact of a gross misplacement of steel with the comment:

> This is a most unfortunate argument to be made by one engaged in the commercial promotion of concrete buildings. If the system of design is so delicate that construction errors by skilled if not expert workmen, foremen and engineers, honestly endeavoring to follow design, can cut in half a computed strength, the design is obviously not proper for engineering construction.

Why is it an unfortunate argument? Steel out of position is an unfortunate, yea, a deplorable, circumstance, but it is a legitimate explanation of the weakness exhibited by the wall panels. Moreover, I am not engaged in the commercial promotion of concrete buildings. I am a consulting engineer engaged in private practice without any connection with the owners or agents of a patent. In the case of the floor in the Bell Street warehouse, I selected the "Mushroom" system upon what I deemed its merits and I offer no apology for the choice.

I cannot reconcile your insinuation that the system of design used is "delicate" with my experience in construction. It may be true that the placing of steel over the interior columns is simpler than over the wall columns, but in neither is it difficult. It requires care, but care is the chief attribute in any vocation. If we are to discard care in construction we must declare 90% of all designs too delicate to be entertained. Good concrete can be made by the exercise of care despite the fact that bad concrete has been mixed by the elimination of care. The observation applies with equal force to the placing of steel reinforcement. No one is justified in asserting, because the steel in the Bell Street warehouse was misplaced, that the position of the reinforcement is not dependable in the "Mushroom" type of buildings. This is the only instance brought to my attention where steel has been misplaced in a Turner floor system. There are probably others but in no greater proportion than occur in all types of reinforced-concrete designs. They constitute unfortunate facts but should not be construed as objections to the designs.

I firmly believe you will, after further deliberation, modify your views regarding the delicate character of the Bell Street warehouse design so far as concerns its capability of erection, and you will admit that the design must not be censured because there was an error in construction.

To my mind, the two chief lessons taught by the Seattle tests are the necessity for care in inspection and the folly in denying to the designer any authority over erection. A dual responsibility should be avoided.

A. O. Powell.

Central Building, Seattle, Wash., June 3, 1916.

[We apologize for the use of "commercial," although the word was not used with derogatory intent. Capt. Powell's discussion included correspondence by representatives of a commercial system (by that meaning a system the subject of barter and sale) which accounts for the confusion.

As to the latter part of Capt. Powell's criticism we can only say that the whole case hinges on what is due "care." The Seattle authorities claim that experts were employed in the warehouse construction although they were not the employees of the designer or of the promoter of the particular system used. Misplaced steel is so common in reinforced-concrete work that many types of design have special precautions to prevent it. A system which permits experts honestly trying to follow design so to disarrange the reinforcement as to cut strength in half lacks something of perfection.

In his last contention, Capt. Powell has our complete agreement. The design of an engineering structure implies its supervision. Anything else is highly undesirable—Editor.]

Deep-Well Propeller Pump with Direct Pumping

Sir—I noticed with considerable interest an article in your issue of June 1, 1916, p. 1038, describing the deep-well pumping system at Memphis, Tenn. It is said that Mr. Wills, the designer and inventor of this system, has obtained a patent on a combination propeller and turbine pump. I also read your comment on his claims and a reference to previous publications on systems of this kind. That Mr. Wills has developed and put into operation a system of this sort is very interesting to me, for the reason that in 1903 I invented a similar pump and had a drawing of it made. It was a combination of the turbine pump manufactured by the Byron-Jackson Co. and the propeller pump manufactured by P. K. Wood. In the summer of 1903 I read at the Municipal League Convention held at Appleton, Wis., a paper on "Pumping Plants for Small Cities," which was published in *The Municipality* for December, 1903. I quote from page 96 of that publication:

None of the pumps so far manufactured are capable of lifting large quantities of water from a comparatively small well and forcing it up into the elevated tank or standpipe, except when the well will supply the pump under ordinary suction or the pump is placed in a pit.

It has been the writer's desire to secure such a pump, if possible, to take large quantities of water from wells and raise it into the elevated tank or standpipe, so as to avoid the additional expense of pits, surface reservoirs and service pumps. The Jackson compound centrifugal pump will work against a pressure of over 100 lb., while the P. K. Wood propeller pump, with certain modifications, will serve nicely for raising the water from the well to the surface.

The writer has conceived the idea of combining these two pumps together upon one shaft so as to raise the water with one pump directly to the standpipe or reservoir. The guides of the Wood pump would be modified and the general mode of construction improved upon. A 1,000,000-gal. pump of this type could be manufactured and placed in a proper well for about $2,000 and a 1,500,000-gal. pump for $2,500. With such a type of pump it would be possible to secure supplies for many of our Wisconsin cities that at the present time are short of water or are using water of inferior quality.

If proper arrangements can be made for the manufacture of these pumps, I am satisfied that they will revolutionize the water-supply problems of this state and others where there are artesian areas. With this type of pump it would be possible to put the wells as far apart as desired and operate a number of these pumps by electric motors from a central power station.

By this plan it would make it possible to secure artesian supplies for many cities of this state, of 20,000 population and over, at a very reasonable figure as compared with the cost of sinking shafts, intermediate reservoirs and two sets of pumps.

Besides this previous design and conception of such an installation there are pumps of this type made by the Indiana Pump Co., and several of them have been sold and are in operation in Wisconsin. The American Well Works, of Aurora, Ill., has such a plant in operation at Rockford, Ill. I cannot exactly see how the Patent Office could have overlooked the previous design and conception of such an outfit and granted Mr. Wills his patent. However, I am glad to know that such a system is being used extensively; and as intimated in my article of 13 years ago, I hope other cities will make use of it.

Madison, Wis., June 7, 1916. W. G. KIRCHOFFER.

Oklahoma City Water-Supply Projects, 1910 and 1916

Sir—On May 20, 1916, the people of Oklahoma City voted more than two to one for the adoption of the same plan of water-supply for the city the development and advocacy of which five years ago nearly cost the writer his life, due to the prejudice created by the circulation, by those holding high state office, of statements like the following:

By all means vote against the bonds Tuesday and protect your loved ones from this infamous scheme to dam up a big pond of deadly disease germs.

These New York engineers may give us liquid sewage at $1,250,000, and then they may retire to their beautiful homes on the banks of the Hudson River and drink from their own pure water-supply, but we will be left behind to bear the burdens and care for the dying and bury the dead and some of us are quite as much interested in protecting the health and lives of the citizens of Oklahoma City as are certain engineers in making a per cent. on a million and a quarter of our money.

I am perfectly willing, personally, and I am a taxpayer, to pay $20 a year for 40 years for clean wholesome water, but not one cent have I for sewage and filth and while I live it shall be over my protest that any engineer, for any amount of money, be permitted to feed liquid sewage to the helpless poor.

I thought of the typhoid epidemics in El Reno, the smallpox cases and social diseases which infect every city. I thought of the sewage from these sick bodies being treated by chemicals and then given to the mouths of our helpless poor to drink.

The facts in connection with the submission of the proposition at that time were clearly set forth in your issue of Mar. 2, 1911, and your editorial comment upon the situation was very much to the point.

Since that time, engineering commission after engineering commission has tried unsuccessfully to secure the adoption of every conceivable plan other than the one rejected at that time.

So bitter, however, was the feeling engendered five years ago by the interference of state officials in the affairs of the city that even at this late date in the campaign that has recently ended successfully for an additional water-supply, those advocating the water bonds were forced to declare that the plan presented was not the Potter plan. An examination of the plan submitted by the writer in 1910 and the plan upon which a favorable vote was recently secured is convincing proof that the present plan and the one advocated five years ago are one and the same, with the exception that the plan proposed by the writer included a method of water treatment in addition to that of simple storage to reduce the permanent hardness with which the waters of the North Canadian are highly charged. ALEXANDER POTTER.

50 Church St., New York City, May 26, 1916.

Subaqueous Tunnel Records

Sir—Referring to the article in your issue of June 1, entitled "Subaqueous Tunnel Record," in which it is stated that claims have been made that the progress in driving the Dorchester Rapid Transit tunnels under the Fort Point Channel, Boston, Mass., established a record for subaqueous tunnel driving, I beg to call your attention to the following records of progress made in constructing the tunnels under the North River for the Pennsylvania R.R. and Hudson & Manhattan R.R.; Charles M. Jacobs, of this firm, being chief engineer for both projects.

The best progress made on the North River tunnels of the Pennsylvania R.R. was in the month of June, 1906, when the North tunnel, driving from Manhattan, made 545.2 ft.

On the Hudson & Manhattan tunnels, the best progress was as follows: North tunnel, Downtown System—the progress for 3 mo., from Apr. 30, 1906, to July 31, 1906, was 2,321.5 ft. The best progress for a month was in this tunnel, in July, 1906, when 967.5 ft. was made.

The best progress for a week was in the South tunnel, from Aug. 8 to 15, 1906, when 308 ft. was constructed. The best progress for one day was on Aug. 1, 1906, when 36 rings, two feet long, were erected, making 72 ft. of tunnel for the day.

Conditions are so different in various localities that no comparison that means anything can be made. In the case of tunneling in the Hudson River silt in the vicinity of New York, the tunnels were advanced by forcing the shield through the silt without any excavating in advance of the diaphragm, and by only admitting through the shield doors a small portion of the material displaced by the shield.

Perhaps a fairer comparison of work accomplished would be had on the basis of volume of material removed or displaced in constructing a tunnel in a given time.

JACOBS & DAVIES, INC.,
George D. Snyder, Director.

30 Church St., New York City, June 6, 1916.

Location Survey Methods for Irrigation Canals

Sir—After reading the article under the above caption (*Engineering News,* May 25, 1916, p. 997) I am constrained to say with Solomon: "Verily, there is nothing new under the sun." As far back as 1877, while connected with the Pittsburgh Southern R.R.—a narrow-gage line between Pittsburgh and Washington, Penn.—I used the identical method, described in the article, in locating the line in crossing three summits.

A preliminary survey was first made of the line between Castle Shannon and Wrights Farm, Penn., a distance of about 14 mi., and the plan was to plot this line, project the paper location and run it on the ground in the usual way.

The work had been let and the contractors were waiting to begin operations. It then occurred to the writer to plot only the "summit" portions, project a few curves and let it go at that.

These "summit" crossings were then run in, only a few hundred feet on each side, and as the grades were 3%, grade contour lines corresponding to this grade were run in on the ground with chain and level, setting stakes 50 ft. apart, anywhere from 500 to 1,500 ft. or more ahead. Proper curves and tangents were then run in to fit the grade contours, curves up to 16° and more being used.

The writer got so accustomed to this work that it only required a look at the staked line to tell almost immediately the proper curvature to use. Of course no compensation was used as the summit crossings were nearly "all curves" and it would have been a refinement to have done so.

The line on the remainder of the railway was run in by the "practical eye," and the methods used saved several months. A portion of this old railway is used by the Baltimore & Ohio R.R. for its Western division.

Buffalo, N. Y., May 27, 1916. EMILE LOW.

Credit for Novel Power Plant

Sir—I have read the article appearing on page 740 of the April 20, 1916, issue of *Engineering News,* "Impulse Wheels Developed for Silt-Laden Water," by J. W. Swaren. In reading this article carefully, I find that it might seem to give the credit for the design of this most interesting power plant, to the Pelton Water Wheel Co. This is not correct, however, and I sincerely trust that you will publish this letter in an early issue, to correct any misapprehension that may exist.

The original conception to use, in this power plant, Pelton wheels operating on vertical shafts, was the work of O. H. Ensign, at that time Chief Electrical Engineer of the United States Reclamation Service. Mr. Ensign, against much opposition, determined on the plan of using Pelton wheels with vertical shafts in place of pressure turbines in this plant, owing to the unfavorable characteristics of the water. Mr. Ensign also worked out all of the complex details of reinforcing that entered into the construction of the plant.

The plan of using Pelton wheels was submitted to the Pelton Water Wheel Co., and their part in the plan consisted in working out the engineering details to adapt a Pelton type of wheel to this particular service, also working out the details for governing control and mechanical structure.

I appreciate that the great success of this plant reflects credit not only on the Pelton Water Wheel Co., which executed the contract, but on Mr. Ensign for his conception of the general plan and the execution of the details of reinforcing. Further than this, it was Mr. Ensign who proposed the very ingenious method of producing a load on the plant by reversing one unit operating as a synchronous motor, with the water projected against the buckets, producing a hydraulic brake and an ideal means for providing load to a hydro-electric unit. There is an error on page 744, second column, the end of line 4; namely, the water strikes *within the bowls of the buckets* as in normal use, and not on the backs of the buckets. This system of providing a load for testing purposes on the hydro-electric plant is ideal, most ingenious, and why it has not been used before it is impossible to explain. In carrying out many important tests, I have entirely overlooked the possibilities of reversing the water wheel and converting it into a water brake, and I have constructed elaborate and expensive rheostats which are more or less uncertain and unsatisfactory, whereas, with the water brake developed by rotating the wheel in the reverse direction and against the impinging jet, Mr. Ensign produces a water brake that can be controlled to the utmost degree in securing an absolutely uniform load for testing.

The Pelton Company would therefore appreciate your publishing these facts so that Mr. Ensign, now Consulting Engineer to the United States Reclamation Service, will receive proper credit for this work, inclusive of this system of testing. W. A. DOBLE,

Chief Engineer, Pelton Water Wheel Co.

San Francisco, Calif., June 10, 1916.

Wanted: Names of Engineers Experienced in Concrete

Sir—The Portland Cement Association is endeavoring to compile a list of names and addresses of engineers specializing in concrete design and construction, for the following purposes:

1. To use in replying to requests for names and addresses of men competent to (a) Design a structure; (b) pass upon a design; (c) report upon a project; (d) report upon a failure.

2. To enable us to give to parties who ask for assistance the names of specialists, for we do not encroach upon the field of men in practice.

3. To enable us to give to specialists in concrete work the names and addresses of parties in their own and near-by cities who are contemplating building and who may be induced to use concrete.

PORTLAND CEMENT ASSOCIATION,
By Ernest McCullough,
Chief Engineer Fireproof Construction Bureau.
111 West Washington St., Chicago, June 13, 1916.

To Clean a Brick Water-Supply Conduit, a city official in charge wishes to make use of mechanically operated brushes mounted on a moving car. The conduit is 5½x6 ft. in cross-section and 4½ mi. long and its inner walls are covered with silt, mixed with more or less slimy vegetable matter. This covering can be removed by hand brushes, but information regarding something cheaper and more expeditious is desired.

Washout at Abutment Drops Train from Bridge

A washout at the Flood Creek bridge on the Chicago, Rock Island & Pacific Ry., at Packard, Iowa, early in the morning of June 2, caused a train wreck and the loss of 13 lives.

A heavy storm occurred in the night, and the water was very high when the train crossed the bridge. The engine, tender, mail car and baggage car crossed in safety. The smoking car was derailed as it approached the bridge and evidently bunched the ties and tore up the guard rails, but did not go off the structure, though it swung round at an angle to the track. The day car went over into the water and was swung round by the current. The rear car (a Pullman) remained on the bank, with its front end over the first span.

The bridge (built in 1896) had three deck-girder spans of 42 ft. The abutments were of stone masonry, with footings in the bank and having no wing walls. The piers were of peculiar construction. Pile foundations carried concrete footings 4 ft. deep, of natural cement, finished with a stone coping. The original intention was to put steel bents on these copings, but instead brick piers were built, about 12½ ft. high, 21 in. thick in the lower portion, but widening at the top for the bridge seat.

The immediate cause of the accident is believed to have been a washing out of the fill just behind the abutment, as the engineman reported feeling a drop at this point. He attributed this to the dropping of the abutment, but in that case it is not likely that the engine and first cars could have crossed in safety.

When the smoking car reached this point, the drop was evidently sufficient to derail it, and it was pulled

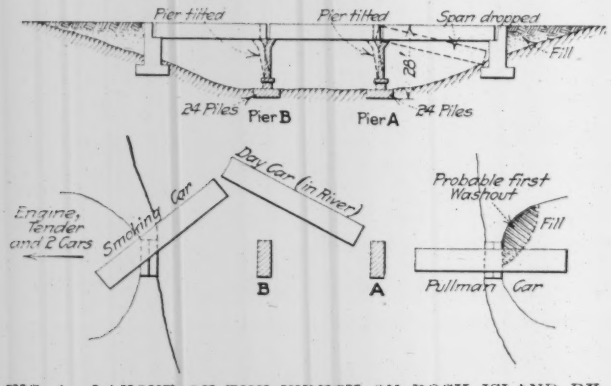


FIG. 1. LAYOUT OF THE WRECK ON ROCK ISLAND RY.

across the bridge, smashing and tearing up the deck and bending the laterals. When it reached the far abutment, it was stopped by the heap of ties it had bunched in front of it. As it stopped, it swung round, throwing the end of the next car off the bridge. The guard timbers were secured by hook bolts in every third tie and were boxed out over the ties. There were no inside guard rails.

The pounding of the engine and cars at the low point drove the entire superstructure forward about 3 ft., with the piers in an inclined position. The abutment was washed out and the end of the first span dropped, but it is believed that this happened at the end rather than at the beginning of the accident.

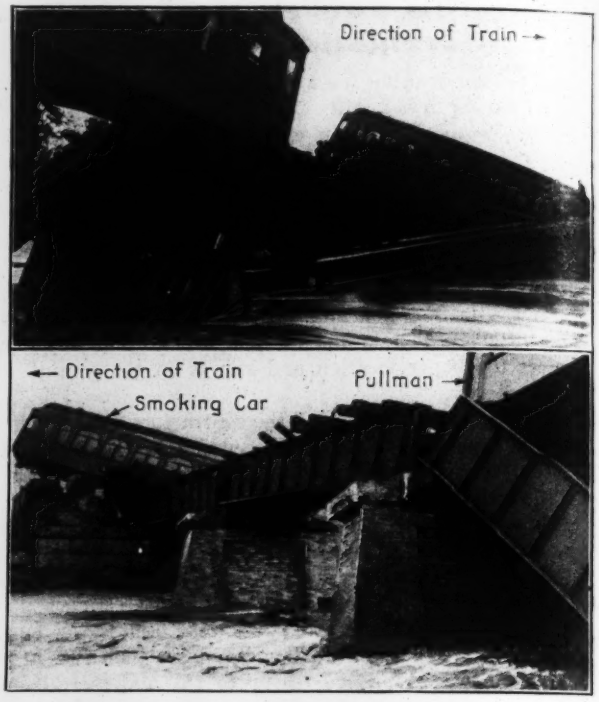


FIG. 2. VIEWS TAKEN AFTER THE RAILWAY ACCIDENT AT PACKARD, IOWA

Owing to the reports of high water at various places, a work train was sent out about half an hour before the passenger train. A stop was made at the Flood Creek bridge, but there was no sign of failure; and as the stream had caused no trouble since the bridge was built, no trouble was expected. Probably at that time the backlash from the strong current was already cutting into the fill behind the abutment, but below the water surface.

After the accident the middle span was removed by wrecking cars. Pier *A* then fell, but pier *B* seems to have returned nearly to its original position.

The upper view (on the upstream side of the bridge) shows at the left the Pullman car spanning the gap over the fallen girders of the first span, and with its truck dropped upon the girders. This was the rear car of the train. In the stream, with the rear end swung under the second span is the day coach, in which all the casualties occurred. Beyond the third span and on the bank is the smoking car, thrown across the track. This was the first car derailed. It jumped the track as it entered the bridge and tore up the floor as it ran across.

The lower view (on the downstream side) shows the two brick piers leaning forward, and torn from the coping (submerged) of the masonry base. The corbelling of the upper part of the piers is indicated, but the arched openings provided for on the plans were evidently omitted and the center wall of each pier built up solid.

Fighting City Smoke Nuisance—The Civic and Commerce Association of Minneapolis has adopted a unique plan to contend with the smoke nuisance. It will issue a "Smoke Manual," a booklet setting forth the city smoke ordinances and information relating to smoke prevention as evolved by the best engineering talent. A daily check will be kept of building permits, and prospective builders of factories and office buildings will be sent a copy of the booklet.

Boston Tech Opens New Home

The Massachusetts Institute of Technology celebrated last week its 50th anniversary and also the completion of the new buildings which it is to occupy with the opening of the new college year. These new buildings, which have been under construction for the past three years, are far more spacious than those of any other engineering educational institution in the world. They have been described in *Engineering News* at various stages in their progress. The accompanying (Fig. 2) presents for the first time a view of the completed buildings. It is somewhat inaccurate to say completed, however, for, although not shown in the photograph, the tops of derricks and concrete hoisting towers still in operation at the rear of the buildings betray the fact that work is still in progress. It is so far advanced, however, that the unfinished portions will doubtless be ready for use by the time the students assemble in the fall. It may be added that the removal of apparatus from the old buildings in Boston to the new quarters has been in progress for months, and a considerable part of the laboratory equipment is already in position. A detailed study of the ground plan of the buildings and ground, shown in Fig. 1, together with the general view from the Charles River in Fig. 2, will make clear to the reader what is still more evident from a personal examination of the buildings—that the group of buildings are remarkable both for their architectural dignity and for their adaptation to the needs of a school of engineering. The central architectural feature of the buildings is the great dome,[1] 120 ft. in span and nearly 150 ft. above the ground. The buildings that flank it on either side cover so wide an area and the dome itself, at the rear of the great central court, is so far back from the wings which front on the river esplanade that one does not realize its size and height. The double row of massive columns on the portico in front of the dome is a splendid architectural embellishment of the central court. In most buildings of classic architecture the failure to admit sufficient light to the interior is a constant trial to those who occupy the building. In the New Technology the architect has boldly met this problem by filling the spaces between the huge pilasters that form the walls of the building entirely with glass.

As may be seen by reference to the general plan, the buildings are of comparatively narrow width, and as they receive light from both sides through this abundant window space, there should be ample illumination at all seasons, even with the overcast skies of Boston. Artificial light, heat, etc., will be furnished to the entire group of buildings from a power house situated some 600 ft. north from the central dome adjacent to the Boston & Albany R.R. tracks. The stack of the power house may be seen in Fig. 2.

The distribution of the different departments in the group of buildings is shown by the inscriptions in Fig. 2. The buildings are three and four stories in height, and in general the laboratories occupy the lower floors and the drafting rooms the top floor, where additional light is afforded by skylights.

When it was first decided to build an entirely new Technology, a prominent engineer, who is also an alumnus of the Institute, prepared at his own expense an elaborate plan for a single great building of many stories in which to house the Institute. While this plan was not adopted as a whole, one feature of it has been incorporated in the present buildings. That is, the entire group of buildings is under one roof so that in bad weather the students need not go out of doors in passing from one exercise to another.

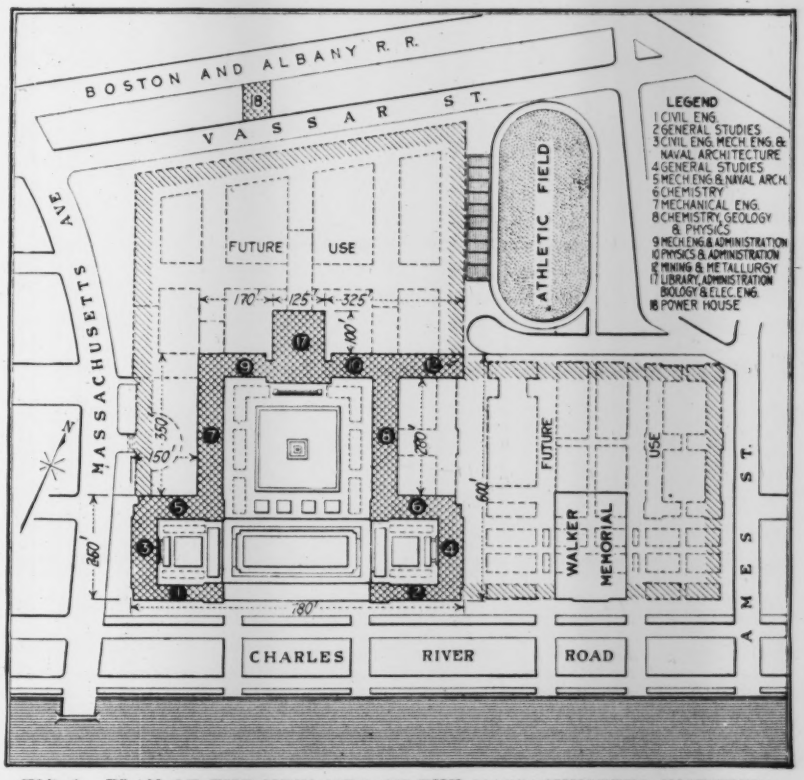


FIG. 1. PLAN OF THE NEW SITE OF THE MASSACHUSETTS INSTITUTE OF TECHNOLOGY

The entire tract owned by the Institute contains about 50 acres, so that, as may be seen from plan (Fig. 1), there is ample room for the Institute to expand its facilities. The only other building at present planned, however, is the Walker Memorial, the cornerstone for which was laid last week. This building is to contain a gymnasium, a students' club, general dining room, dormitory facilities, etc.

Of no small interest in connection with the New Technology is the remarkable manner in which it has been financed. The Institute, like every other great and rapidly growing educational institution, has required all its

[1] "Engineering News," Aug. 20, 1915.

available funds for its running expenses and had no money with which to erect a new plant suited to its needs. While the land which its old buildings occupy in the heart of Boston should eventually yield a large sum, that resource was not immediately available nor was it practicable to use for the new buildings the money in its endowment funds or that which it will eventually receive from the McKay bequest through its coöperative agreement with Harvard University. Under these conditions the money to pay for the land at the new site was contributed by generous Boston alumni, and then President Maclaurin found somewhere a generous benefactor who has contributed the money, some $3,500,000, with which the new buildings have been erected. Only one condition has been placed upon this and that is that no one whatever, except President Maclaurin, may know the identity of the giver.

The prosperous man who builds himself a large new residence generally finds when he moves in that the running expenses of his household have materially increased. It is evident that the Institute in its new home must increase its running expenses if it is to make full use of its greatly enlarged laboratory facilities and pay salaries sufficient to secure high-grade instructors. Professor Maclaurin has therefore appealed for a large addition to the Institute's endowment funds. Again the mysterious benefactor, whom all the Institute knows as "Mr. Smith" for want of a better name, has come forward with the offer, announced at the alumni dinner on Wednesday evening, that he will give $5 to the Institute for every $3 raised by the alumni prior to Jan. 1. He has agreed to make this promise good up to a contribution of $2,500,000 on his part, so that if the alumni can raise their portion, an addition of $4,000,000 should be made to the Institute's endowment. That the alumni are likely to make good their share of the contribution appeared evident at the dinner on Wednesday evening, when gifts were announced from six of the Institute's alumni aggregating a total of $1,000,000.

After the foregoing statement it seems superfluous to add that the alumni of no engineering school in the country take a more active interest in their Alma Mater than do the men of M. I. T., and that the opportunity to celebrate at once the Institute's 50th anniversary and take part in the dedication of the New Technology brought an unprecedented number of former students back to Boston.

Of the various interesting events with which the three celebration days were full, space permits brief mention here of only the two or three most important. The pageant in connection with the removal from the old buildings to the new on Tuesday evening was a ceremony unique in college annals. The officers of the Institute assembled at the old buildings, took the great seal and charter of the Institute, which had been placed in elaborate caskets, and marched in procession down Berkeley St., to the Charles River. Here at a special landing was waiting the "Bucentaur," a reproduction of an ancient state barge, propelled by 40 rowers selected from the Institute's undergraduates. In this quaint vessel the dignitaries were conveyed to the landing in front of the new buildings. In the great central court of these buildings had been erected a grand stand seating 7,000 people. In front of the stand was a throne for Governor McCall of Massachusetts, and lesser thrones for the Mayors of Boston and Cambridge. In the arena in front of the spectators was then enacted a play in which 1,000 actors took part, the play depicting the development of civilization from original chaos to the establishment of righteousness and the liberal arts and the enthronement of education.

Dedicating the New Buildings

When the pageant was over and the audience had dispersed, a huge gang of workmen appeared and all the rest of the night there was a continuous din of hammer and sledge and saw. The next morning not a vestige was left of the huge timber structure which 7,000 people had occupied a few hours before. The entire area in the central court was cleaned and smoothed, and by the afternoon 10,000 chairs had been placed there for the audience at the dedication exercises. The speakers at these exercises besides President Maclaurin, were President Lowell of Harvard, Governor McCall, and Senator Henry Cabot Lodge.

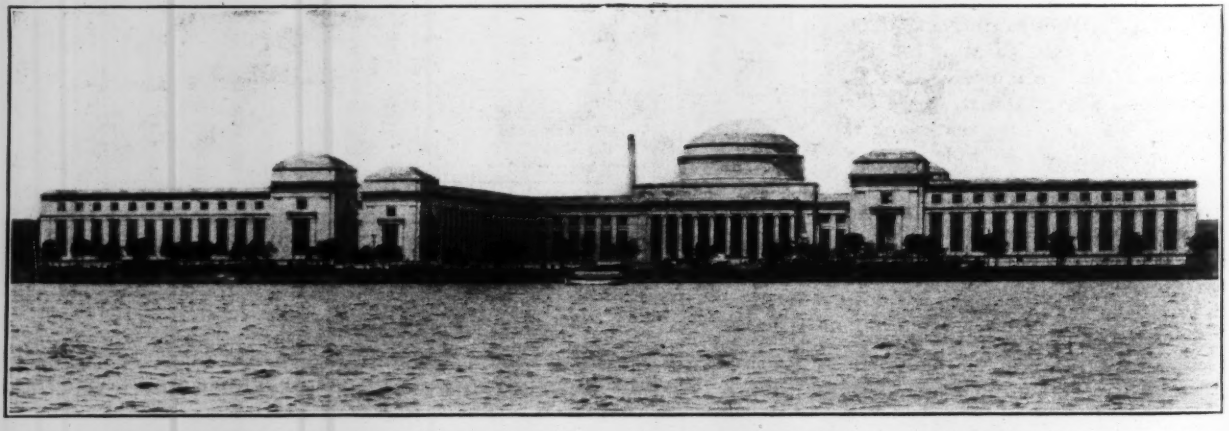


FIG. 2. NEW BUILDINGS OF THE MASSACHUSETTS INSTITUTE OF TECHNOLOGY FROM THE CHARLES RIVER

In view of the coöperative agreement between Harvard and Technology, the following excerpt from President Lowell's remarks is of especial interest:

The Massachusetts Institute of Technology and Harvard University have agreed to coöperate in engineering and mining education in order by so doing to render together a greater service to the public. By this spirit we have shown that we are more quick in our sense of public duty than some state institutions, which, although paid from the same treasury, maintain a needless rivalry at the public's expense.

The dominant note in Governor McCall's address and also in that of Senator Lodge was that education in the technological school as well as in the university must be spiritual and not materialistic in its ultimate aim if it is to be worth while. Senator Lodge emphasized especially the duty owed by college-trained men to their country, a duty important at this particular juncture. The following brief quotations are of especial interest:

> The great lesson which, to my thinking, should be learned here is that education and knowledge are not ends in themselves, but means to an end; and that one great purpose to be here achieved is to go forth with the understanding that all who have these privileges are units in the making of a nation. Our learning is vain if it fails to teach us that nations, like men, must have a conscience and a soul. . . .
>
> We draw our blood and traditions from people and many lands, but we labor in vain if here on this soil we are aught but Americans with one country, one loyalty, and one tradition common to us all. . . . I have seen in the newspapers that the Anti-Enlistment League is preparing to circulate a pledge in our colleges and universities which runs in this wise:
>
> I, being over 18 years of age, hereby pledge myself against enlistment as a volunteer for any military or naval service in international war, offensive or defensive, and against giving my approval to such enlistment on the part of others.
>
> In the early days when our independence was won, in the later days when our Union was saved, such a pledge would have been described by the ugly word "treason." That anyone should now think such a pledge possible is an evil sign. People who will not make ready to defend their own peace, their own security and their own honor are well on the way to hopeless war or to supine submission.

FIGS. 3 AND 4. VIEWS OF THE "BUCENTAUR" BEFORE AND AFTER LAUNCHING

The state barge on which the officers of the Institute, bearing the charter and great seal, were conveyed across the Charles River from the old buildings of the Institute to the new

Alumni Banquets in 35 Cities Joined by Telephone

The crowning event of the celebration occurred at the alumni banquet, when the greatest feat in long-distance telephony performed was accomplished. In every place at the dinner tables in Symphony Hall, where the banquet was held, and in every chair in the galleries, there was connected a telephone receiver, some 3,000 in all. According to a prearranged plan, at the same hour at which the banqueters were assembled in Boston, there were gathered, in 35 other cities of the Union, alumni of the Institute and guests, and at each of these assemblages everyone present was provided with a regulation telephone receiver.

For two hours, from nine o'clock to eleven, Eastern time, the gatherings in every one of these cities were continuously connected. Every word spoken at the Boston dinner was heard by the diners in the 35 other citites, including such remote points as New Orleans, Los Angeles, San Francisco, Seattle, Spokane and Anaconda.

As the telephone connection was established, the toastmaster, Charles A. Stone, placed the exercises in charge of John J. Carty, President of the American Institute of Electrical Engineers and Chief Engineer of the American Telephone and Telegraph Co. Mr. Carty in turn put the telephone demonstration in the hands of A. W. Drake, of the class of '95. Mr. Drake called the roll of each of the cities in turn and received from each a reply, in which the speaker gave his own name and class, stated where the party was assembled, and the number of alumni and guests present. Every city responded as promptly when its name was called as though the speakers answering were seated at an adjacent table. Of course these conversations were heard through the telephone receivers not only by the audience of 3,000 at Boston, but by all those listening at the receivers at the dinners held in the 35 other cities. Following this "calling of the roll," President Maclaurin addressed the alumni, and then followed addresses by Dr. Michael I. Pupin, the inventor of the loading coils which solved the difficult problem of long-distance telephone transmission; Alexander Graham Bell, the inventor of the telephone; Orville Wright, the inventor of the aëroplane; Dr. Henry N. Pritchett, President of the Carnegie Foundation and former President of the Institute; and Coleman du Pont. The diners also listened to a conversation between Professor Cross of the Institute, present at the Boston banquet, and Professor Harris J. Ryan, in San Francisco. Then President Lowell of Harvard talked with Dr. H. P. Judson, of the University of Chicago, in Chicago. The famous Tech cheer was given in unison, first by a group of cities in the North, then by a group in the South, then a group in the West, then in the East, and then by all of them together. The diners at Milwaukee then joined in singing the Stein song for the edification of all the rest. As a grand finale to the demonstration Washington sang the Star Spangled Banner, while all the cities joined in the chorus.

As is well known, the exhibition of transcontinental telephone transmission with receivers connected to all the seats at a public dinner has become fairly common, but the connection of such an enormous telephone network, enabling such a large number of people scattered in so many cities to hear everything transmitted through the circuit from any point is a feat never before accomplished. The success with which it was achieved reflects great credit upon the telephone organiz

News of the Engineering World

Bridge or Tunnel for River Crossing at New Orleans

The question of a Mississippi River crossing at New Orleans is once more being agitated, on account of a proposition before the Louisiana legislature. A report has recently been made to the Mayor of New Orleans by the engineering firm of Ford, Bacon & Davis, comparing the economical and engineering factors surrounding the construction of a bridge over the river, which is about 2,300 ft. wide at the site selected, or a tunnel under the river. This report, which is only preliminary, states that a bridge, similar to the one suggested in the 1893 project for a crossing here, would cost $8,000,000 for a railroad line alone and $13,000,000 for full provision for railroad and vehicular traffic. The estimate on the tunnel is $5,000,000 per single-track tube, the suggestion being made that the Detroit River method would be admirably adapted for the mid-river section.

Chicago Centralizes Work of Waste Disposal

The Bureau of City Waste Disposal has been established as a part of the city government at Chicago, to handle all matters in regard to the collection and disposal of garbage and refuse. It will concentrate work and responsibilities formerly divided between the Department of Health and the Department of Public Works. Under the ordinance, the head of the Bureau will be Henry A. Allen, Mechanical Engineer in charge of the garbage plants, who retains this position. He will have entire charge of the design, construction and operation of the new reduction and incineration plants.

Flood Damage in Pennsylvania

Heavy rainfall over the entire watershed of the Susquehanna River in Pennsylvania for a period of 10 days has swollen the waters of that stream to flood stage and caused much damage. At Lawrenceville, Tioga County, several small dams broke, inundating the town to a depth of 4 ft. There were no fatalities, but to take care of sanitary conditions the State Health Department dispatched two medical officers, two sanitary engineers and a corps of health officers from nearby counties to the scene. Breaking of a main gas line between Elmira, N. Y., and Corning, N. Y., left the town without light June 17. The break has been repaired.

At Lewistown, Penn., the Juniata River reached a stage of 21.6 ft. above low water at midnight, June 18. Several earth slides blocked highways and flooded cellars, causing people to move to upper floors. Near East Waterford, Juniata County, on June 17 the swollen waters of Tuscarora Creek washed away one abutment of a small bridge on the Tuscarora Valley R.R. Three cars and a locomotive dropped into the gap; the engineer was killed and the fireman fatally injured.

On the Central division of the Pennsylvania R.R. several landslides tied up traffic. On the Renovo division a slide at Lock Haven tore up tracks for 200 yd. At Spruce Creek, a 10-ton slide derailed a fast passenger train—no casualties.

At Harrisburg, the flood crested at 16.2 ft. at 6 p.m., June 18. Work was suspended on the new Cumberland Valley R.R. bridge and carried away several sections of the temporary trestle. One man was injured in endeavoring to secure the trestle.

New School of Sanitation

A School of Hygiene and Public Health will be established in Baltimore by the Rockefeller Foundation "in connection with and as an integral part of Johns Hopkins University." The central feature of the school, which it is expected will be opened in October, 1917, will be an Institute of Hygiene. A building will be erected near the University hospital. To house various laboratories and departments, including those of sanitary chemistry, bacteriology, epidemiology, industrial hygiene, vital statistics, a museum and a library. Additional facilities for instruction and research will be afforded by the existing schools of medicine, engineering, social science, etc. The Director of the School will be Dr. Wm. H. Welch, Professor of Pathology at Johns Hopkins. Among the experts consulted by the Rockefeller Foundation in formulating its plans for the establishment of the School were the following engineers and sanitarians well known to the readers of *Engineering News:* D. D. Jackson, Edwin C. Jordon, Dr. M. J. Rosenau, Geo. C. Whipple and C.-E. A. Winslow. Others who were included in the conference were Dr. A. C. Abbott, formerly Chief of the Bureau of Health of Philadelphia; Dr. Herman M. Biggs, Commissioner of Health of New York State; Dr. Wm. H. Park, of the New York City Department of Health; and Doctors Simon Flexner, Victor G. Heiser, Theobald Smith and Wm. H. Welch.

San Francisco to Build Weir for Flood Prevention

By E. A. Bailey*

A contract for furnishing all materials and constructing a concrete weir, with accompanying highway and railway bridge structures, was awarded to Teichert & Ambrose, San Francisco, Calif., on June 10, 1916, by the City of Sacramento, the contract price being $330,640.

This structure is known as the "Sacramento Weir," and is a portion of the Sacramento Flood Control Project, reported on in 1910 by the California Débris Commission of Federal Engineers (House Document No. 81, 62nd Congress, 1st Session).

*Flood Control Engineer, State Reclamation Board, Sacramento, Calif.

In the absence of appropriations by the State and Federal Governments for the construction of this weir, the City of Sacramento, being most vitally interested in its construction, voted bonds in 1912 for that purpose, as well as for the construction of the Sacramento Bypass. This bypass is 1¾ mi. long and leads from the weir to the proposed Yolo Bypass in Yolo Basin in order to relieve the Sacramento River of its excess flood waters, which are due to the addition of the flood waters of its tributary, the American River, immediately above the City of Sacramento. As later investigations have shown, these flood waters combined may reach a total quantity of 298,000 sec.-ft., or 2.7 times as much as could be carried by the Sacramento River channel, averaging 750 ft. in width, past and below the City of Sacramento.

The later investigation referred to resulted in an enlargement of the plans for the Sacramento Bypass and Weir, and the confining of the city's funds to the construction of the weir, leaving the construction of the bypass to the State Reclamation Board, which board is now causing an assessment to be levied upon all lands to be benefited to provide funds therefor.

The weir, as now planned and approved by the Chief of Engineers, United States Army, is to be 2,000 ft. in length between abutments, with its permanent sill at El. 25 (practically the elevation of the natural river bank before the levee, which is to be removed, was built at this point), but with a removable crest at El. 31, different panels of which can be either tripped by hand or automatically by floats, as the water rises to successive stages, all being open for the maximum flood discharge of 188,000 sec.-ft. when about El. 37 is reached by the flood plane.

The removable crest is to be of the needle type, and can be replaced as soon as the water has fallen below El. 31, so as to hold as much as possible of the subsiding flood within the river channel to keep it thoroughly scoured out.

The contract for the weir includes the construction of a barge fitted with gates which can be lowered to bed of channel. If necessary, this barge can be floated into position covering the opening between any two piers of the highway and railroad bridges, which, for economy, are to be constructed in conjunction with the weir, thus cutting off the flow through any panel while the removable needles are being replaced.

This plan of providing weirs and bypasses is in direct opposition to the plans for flood control on the Mississippi River where the current recommendations are against outlets.

There are two essential points of difference in conditions which should be noted as accounting for the adoption of relief weirs for the Sacramento. First, the duration of the Sacramento floods are short, lasting but a few days, with a maximum flow of but a few hours instead of a long-continued high flow, as in the Mississippi; and second, the ratio of excess for the Sacramento is from 1.7 to 6 times the channel capacity, the channel being comparatively narrow and easily scoured, during the subsidence of a flood, by the closing of gates on the outlet weirs; while the corresponding ratio for the Mississippi is only a fraction of its channel capacity, which channel is very wide and in much greater danger of silting up from a diversion of a part of its flow.

Concrete Now Permitted for New York City Pavement

Concrete pavements may be laid in New York City under certain conditions, authorized by a resolution of the Board of Estimate and Apportionment, adopted May 26. According to the charter of the City of Greater New York, street pavements paid for wholly or in part by assessment are divided into two classes, permanent pavements and preliminary pavements. The Board of Estimate and Apportionment has the authority to place pavement types in either class. Those designated permanent pavements are the following, when laid on a base of 6 in. or more of portland-cement concrete: Stone block, wood block, 3-in. asphalt block, 3-in. sheet asphalt, vitrified brick and iron slag blocks. The pavements designated preliminary are stone block on sand foundation, 2-in. asphalt block, sheet asphalt on a 4-in. concrete foundation, bituminous concrete, bituminous macadam, and since the resolution above referred to, waterbound macadam and concrete. The latter two were adopted on request of two of the Borough highway departments, and it is a part of this resolution that these types of pavement are to be used only under conditions prescribed by the Board of Estimate and Apportionment in each particular case.

It is not the intention of the city to use either type on heavy traffic streets but only in undeveloped districts where the conditions are similiar to those on country roads.

Tugboat Shifts New Drawbridge—A recently-built steel draw over the Blackwater River at Milton, Santa Rosa County, Ala., was struck by a tugboat and knocked out of position. The span was shoved over to the edge of the pier.

A Disastrous Dust Explosion in No. 3 grain elevator of the Pennsylvania R.R. at the Canton terminal at Baltimore, Md., destroyed the structure and the 1,000,000 bu. of grain it contained, and set fire to a 700-ft. pier and two steamships, contained, and set fire to a 700-ft. pier and two steamships on June 13. The loss amounted to five lives and $2,000,000.

The Water-Power Development Bills now pending in Congress will probably be taken up and pressed for passage as soon as the appropriation bills are out of the way, according to a recent statement by Representative Adamson of Georgia, Chairman of the Committee on Interstate and Foreign Commerce. Judge Adamson has drafted a substitute for the Shields water-power bill, which was passed by the Senate.

The Cost of Making Gasoline from Fuel Oil by the Rittman process is made public in a bulletin just issued by the Bureau of Mines. The first cost of a plant with a capacity for producing 850 bbl. of gasoline per month, or 35,700 gal., is estimated at $20,000. Where fuel oil can be purchased at 50c. a barrel, the estimated cost of the gasoline produced, including interest and depreciation, is 7.8c. per gal. With fuel oil at $1 per bbl., the estimated cost of the gasoline produced is 9.74c. per gal. With fuel oil at $2.10 per bbl., the estimated cost of the gasoline produced is 13.9c. per gal.

A Coal-Handling Wharf Collapsed in Portland, Me., harbor on June 9, and wrecked a part of the handling apparatus. Several workmen who were on it at the time managed to escape uninjured because of the ample warning of collapse which was given. The 120x1,200-ft. wharf, which was comparatively new, consisted of a central fill with granite retaining walls on timber piles. It carried at the shore end bins containing about 10,000 tons of coal and along one side a substantial timber trestle with cable railway connecting to the outer end of the wharf. There was also another trestle carrying two unloaders. The timber piles penetrated a soft clay stratum some 80 or 90 ft. deep but did not go through it to the hard sand beneath. The immediate cause of the collapse was apparently the racking of the wharf by a large steamship which collided with the corner of the wharf during a heavy gale. The piles carrying the nearby trestle tipped to a batter of 1 to 4 and the piers carrying the unloader trestle subsided noticeably. One of the unloaders was wrecked and a large part of the wharf wall slid into the dock.

Engineers Are to Manage a Club in New York City—The Automobile Club of America has retained the J. G. White Management Corporation to operate and manage the club house, which extends from 54th to 55th St., west of Broadway. The club operates garages, testing laboratories, machine shops, a supply department, an efficiency bureau and a bureau of tours, and issues various publications. All these are to be in charge of the Management Corporation.

Elimination of Toll Bridges spanning the Delaware River between Pennsylvania and New Jersey is being considered by two commissions representing the two states. The first meeting of the Pennsylvania commission was held at the Capitol, Harrisburg, Penn., on June 13, when Willis Whited, Engineer of Bridges for the Pennsylvania State Highway Department, was designated to coöperate with an engineer to be named by the New Jersey commission in ascertaining the value of the 16 bridges under consideration. These engineers will report at a joint meeting to be held later in the summer.

Seventeen Flood-Prevention Reservoirs on the drainage areas of the Allegheny and Monongahela Rivers in Pennsylvania, Maryland and West Virginia at an ultimate cost of $20,000,000 were discussed at a conference between Governor Brumbaugh, of Pennsylvania, and representatives of the City of Pittsburgh, Allegheny County and the Flood Commission of Pittsburgh on June 16. Governor Brumbaugh asked the delegates to send him drafts of a bill designed to bring about coöperation between all the authorities whose aid in the enterprise must be enlisted. In the delegation were N. S. Sprague, E. K. Morse and Morris Knowles.

PERSONALS

Charles E. Hewes, City Engineer of Alhambra, Calif., has been made City Manager also.

Andrew F. Macallum, M. Can. Soc. C. E., recently City Engineer of Hamilton, Ont., has been made Commissioner of Public Works of Ottawa, Ont.

Frank C. Worbs has been appointed Assistant Engineer of the Wheeling & Lake Erie R.R., succeeding E. U. Smith, with headquarters at Cleveland, Ohio.

M. M. Marsh has been appointed Superintendent of Ways and Bridges of the Northern Railway Co. of Costa Rica, with headquarters at Siquirres, Costa Rica, C. A.

Henry A. Allen, Mechanical Engineer in charge of the municipal garbage-disposal plant at Chicago, has been made head of the new Bureau of Waste Disposal.

J. O. Halliday, recently Assistant to the General Manager of the New York, New Haven & Hartford R.R., New Haven, Conn., has been appointed Superintendent of Transportation.

Samuel Rea, M. Am. Soc. C. E., President of the Pennsylvania R.R., was awarded the honorary degree of Doctor of Science at the recent commencement at Princeton University.

B. H. Klyce and **Walter J. Kackley,** Assoc. M.'s Am. Soc. C. E., announce the formation of the firm of Klyce & Kackley, Civil and Consulting Engineers, Nashville, Tenn., and Miami, Fla.

C. F. Herington, recently Assistant Engineer in the office of the Mechanical Engineer of the New York Central Lines, is now Mechanical Engineer of the Powdered Coal Department of the Bonnot Co., Canton, Ohio.

Theodore E. Wells, formerly with the engineering bureau of the Pennsylvania State Health Department, is now associated with Farley Gannett, Assoc. M. Am. Soc. C. E., Consulting Engineer, Harrisburg, Penn.

Mortimer L. Meinken, of Brooklyn, N. Y., has been awarded the $100 prize offered by the Barber Asphalt Paving Co. to students of the highway-engineering course at Columbia University for the best essay on roadwork.

Frederick W. Lindars has been appointed to the newly created office of Chief of Accounts of the Public Service Commission, First District, New York. For several years he was with the Bureau of Municipal Research, New York City.

T. C. Mitchell, Junior Engineer, United States Engineer Office, Great Bridge, Va., has been appointed by the Department of State to represent the Liberian Government on the Franco-Liberian Boundary Commission and will leave shortly for Liberia.

Waldemar Lindgren, M. Am. Inst. M. E., Professor of Economic Geology, Massachusetts Institute of Technology, formerly Chief Geologist of the United States Geological Survey, has been awarded the honorary degree of Doctor of Science by Princeton University.

William von Phul, M. Am. Soc. C. E., of Ford, Bacon & Davis, Consulting Engineers, New York City, has been appointed Vice-President and General Manager of the United Railroads, San Francisco, Calif. He is a graduate of Tulane University, Louisiana, and has been associated with Ford, Bacon & Davis since 1907.

William C. Gorgas, Surgeon-General, U. S. A., sailed recently from New York to inspect South American ports. General Gorgas goes as the head of the Yellow Fever Commission of the International Health Board of the Rockefeller Foundation. He has been granted four months' leave of absence from the army.

S. L. Strickler, of Columbia, Penn., has become associated with Farley Gannett, Assoc. M. Am. Soc. C. E., Consulting Engineer, Harrisburg, Penn. Mr. Strickler will have charge of engineering on the new Susquehanna River dam at York, Penn., for the York Haven Water and Power Co. He was formerly Assistant Engineer with F. H. Shaw and with the Pennsylvania and Wabash railroads.

Clifford Shoemaker, former Deputy Highway Commissioner of Construction of the Ohio State Highway Department, and R. C. Watts, also of the Ohio State Highway Department, have formed the Engineering Service Co., Columbus, Ohio. The new firm will take contracts in railroad, highway and municipal construction work. Mr. Shoemaker is Manager and Mr. Watts is General Superintendent.

E. W. Moir, M. Am. Soc. C. E., formerly a partner of S. Pearson & Son, Ltd., London, England, and well known in this country in connection with the Pearson company's work in the construction of the Hudson and East River tunnels of the Pennsylvania R.R., has been made a baronet in recognition of his services to the British Empire. Mr. Moir is at present in this country representing the Ministry of Munitions.

Edmund Blake, whose appointment as Engineer on the construction of the new dry dock at Boston, Mass., for the Holbrook, Cabot & Rollins Corporation was noted in these columns of May 11, was never connected with the Arrowrock dam as stated. Mr. Blake was Manager of the Idaho Irrigation Co. at Boise. The Arrowrock dam was built under the immediate direction of Charles H. Paul, Construction Engineer. James Munn was Superintendent of Construction. A. B. Mayhew was principal engineering assistant.

J. C. Hutchins, M. Am. Soc. C. E., has resigned as President of the Detroit United Ry. and has been elected Chairman of the Board of Directors. His early experience was in railway construction with the Missouri, Gulf & Lexington R.R. and other Southwestern lines. In 1894 he was elected Secretary and Treasurer of the Detroit Citizens' Street Ry. He subsequently held the same office with the Detroit United Ry., later being promoted to the position of Vice-President and General Manager, and in 1902 to the presidency of the company, which under his administration has become a system comprising 800 mi. of electric railway.

OBITUARY

Jared G. Yarnell, a retired railway contractor of Reading, Penn., died June 16, aged 76 years. For a number of years he was with the bridge department of the Lehigh Valley R.R.

Hamilton Flood, civil engineer and contractor, died at the City Hospital, Quincy, Mass., on May 23, at 46 years of age. Mr. Flood was the first City Engineer of Quincy, being appointed to the office when it was established in 1902. After serving in that capacity for six years, he went into contracting.

Silvanus P. Thompson, Past-President of the Institution of Electrical Engineers, died at his home in London, England, June 13. He was born in York, June 19, 1851. He was educated at Bootham School at York, Flounders' Institute and the Royal School of Mines. For many years he was Professor of Physics in the City and Guilds Technical College, Finsbury. He was chiefly known as the author of some of the earliest works on electrical engineering.

George Gilmour, M. Am. Soc. M. E., Chief Engineer of the Travelers Insurance Co. of Hartford, died at his home in New York City, June 15, aged 51 years. He was born in

Petrograd, Russia, where his father was engaged as Naval Constructor for the Russian Government. He was educated in Russia and in England, and his early experience was in marine engineering. Previous to his work with the Travelers Insurance Co. he was Mechanical Engineer with the New York Telephone Co. Mr. Gilmour was a trustee and member of the executive committee of the Museum of Safety and in recent years had established a reputation as an expert in accident prevention and industrial safety. He is survived by a widow.

Charles Hopkins Cartlidge, Bridge Engineer of the Chicago, Burlington & Quincy R.R., died of pneumonia, June 14, after a very short illness. He was born at Hannibal, Mo., in 1869 and after a public-school education graduated from the high school at St. Joseph, Mo. His engineering career commenced in 1886, when he went to the Kansas City, Memphis & Birmingham R.R. as rodman. He was with that road for two years and then (in 1889) became county surveyor for Buchanan County, Missouri. In 1890 he entered the service of the Chicago, Burlington & Quincy R.R. as draftsman in the bridge department and remained with that road until his death. In 1902 he was appointed Bridge Engineer and was in charge of all bridge design and construction, this road doing much of its bridgework with its own forces. He was largely instrumental in the introduction of the concrete trestle and in the use of separate or precast concrete slabs as self-supporting spans for trestles and track-elevation subways. His largest work was the design of the great Ohio River bridge now under construction at Metropolis, Ill. This has a double-track simple-truss channel span of 720 ft., which is the longest simple-truss span ever built and is notable for the extensive use of silicon steel in the main members. He was Chief Engineer of the Paducah & Illinois R.R., which is a subsidiary company for building the railway connection of which this bridge is the principal feature. In 1893 Mr. Cartlidge was married to Louise E. Almy, at Oak Park, Ill. His residence was at Hinsdale, Ill. He was a member of the American Society of Civil Engineers, the American Railway Engineering Association, the American Society for Testing Materials and the Western Society of Engineers.

ENGINEERING SOCIETIES

NATIONAL GAS-ENGINE ASSOCIATION.
June 27-29. Annual convention in Chicago. Secy., H. R. Brate, Lakemont, N. Y.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
June 27-30. Annual meeting in Pittsburgh. Secy., Charles Warren Hunt, 220 West 57th St., New York City.

AMERICAN SOCIETY FOR TESTING MATERIALS.
June 27-30. Annual meeting in Atlantic City. Secy., Edgar Marburg, University of Pennsylvania, Philadelphia, Penn.

INTERNATIONAL ASSOCIATION OF FIRE ENGINEERS.
Aug. 29-Sept. 1. Convention in Providence, R. I. Secy., James McFall, Roanoke, Va.

ASSOCIATION OF EDISON ILLUMINATING COMPANIES.
Sept. 4-7. Convention in Hot Springs, Va. Asst. Secy., E. A. Baily, 360 Pearl St., Brooklyn, N. Y.

THE TRAVELING ENGINEERS' ASSOCIATION.
Sept. 5-8. Annual meeting in Chicago, Ill. Secy., W. O. Thompson, care General Offices, N. Y. C. R.R., Cleveland, Ohio.

AMERICAN FOUNDRYMEN'S ASSOCIATION.
Week of Sept. 11. Annual meeting in Cleveland, Ohio. Secy., A. O. Backert, 12th and Chestnut St., Cleveland, Ohio.

THE AMERICAN INSTITUTE OF METALS.
Week of Sept. 11. Meeting in Cleveland, Ohio. Secy., W. M. Corse, 106 Morris Ave., Buffalo, N. Y.

NATIONAL ASSOCIATION OF STATIONARY ENGINEERS.
Sept. 11-16. Convention, Minneapolis, Minn. Secy., Fred W. Raven, 417 S. Dearborn St., Chicago.

MASTER CAR AND LOCOMOTIVE PAINTERS' ASSOCIATION.
Sept. 12. Convention in Atlantic City. Secy., A. P. Dane, Reading, Mass.

RAILWAY SIGNAL ASSOCIATION.
Sept. 12-14. Convention in Mackinac Island. Secy., C. C. Rosenberg, Myers Building, Bethlehem, Penn.

ILLUMINATING ENGINEERING SOCIETY.
Sept. 18-20. Convention in Philadelphia, Penn. Asst. Secy., C. D. Fawcett, 29 West 39th St., New York, N. Y.

ASSOCIATION OF IRON AND STEEL ELECTRICAL ENGINEERS.
Sept. 18-22. Convention in Chicago. Secy., W. O. Oschmann, Oliver Iron and Steel Co., Pittsburgh, Penn.

THE ROADMASTERS AND MAINTENANCE OF WAY ASSOCIATION OF AMERICA.
Sept. 19-22. Convention in New York City. Secy., P. J. McAndrews.

MICHIGAN GAS ASSOCIATION.
Sept. 21-22. Annual meeting in Detroit, Mich. Secy., Clark R. Graves, Lansing, Mich.

AMERICAN PEAT SOCIETY.
Sept. 21-23. Annual meeting in Washington, D. C. Secy., Julius Bordollo, Kingsbridge, N. Y.

THE TRACK SUPPLY ASSOCIATION.
Sept. 19-22. Convention in New York City. Secy., W. C. Kidd, Ramapo Iron Works, Hillburn, N. Y.

RAILWAY FIRE PROTECTION ASSOCIATION.
Oct. 3-5. Convention in New York City. Secy., C. B. Edwards, Mobile & Ohio R.R., Mobile, Ala.

AMERICAN SOCIETY OF MUNICIPAL IMPROVEMENTS.
Oct. 9-13. Convention in Newark, N. J. Secy., Charles C. Brown, Indianapolis, Ind.

AMERICAN ELECTRIC RAILWAY ASSOCIATION.
Oct. 9-13. Convention at Atlantic City, N. J. Secy., E. B. Burritt, 8 West 40th St., New York City.

AMERICAN RAILWAY BRIDGE AND BUILDING ASSOCIATION.
Oct. 17-19. Convention in New Orleans. Secy., C. A. Lichty, C. & N. W. Ry., Chicago.

AMERICAN GAS INSTITUTE.
Oct. 17-20. Annual meeting in Chicago. Secy., G. G. Ramsdell, New York, N. Y.

AMERICAN PUBLIC HEALTH ASSOCIATION.
Oct. 24-27. Convention in Cincinnati, Ohio. Secy., Prof. Selskar M. Gunn, Boston, Mass.

RAILWAY ELECTRICAL SUPPLY MANUFACTURERS ASSOCIATION.
Oct. 30-Nov. 3. Convention in Chicago. Secy., J. Scribner, care General Electric Co., Chicago.

NATIONAL ASSOCIATION OF RAILWAY COMMISSIONERS.
Nov. 14. Convention in Washington, D. C. Secy., William H. Connolly, Washington, D. C.

RAILWAY GARDENING ASSOCIATION.
Dec. 5-7. Convention in New Orleans, La. Secy., Charles E. Lowe, Pittsburgh Lower Western R.R., Sewickley, Penn.

The Cleveland Engineering Society held its annual dinner on June 13 in the main auditorium of the Chamber of Commerce building. The attendance was 310. Gustav Lindenthal, on whom was conferred honorary membership, gave the principal address, entitled "Monumental Bridges." Mr. Lindenthal, designer and builder of great bridges of which the most recent is the Hell Gate Arch, was a charter member of the Cleveland Engineering Society in 1880 and has continued his membership ever since. The address conferring honorary membership was delivered by Frank C. Osborn.

President-elect F. W. Ballard outlined a program of society and civic activity. Plans were announced by him looking toward raising a fund of $200,000 for a home for the society and for a large endowment for the library. J. H. Stratton is vice-president elect. Early in July the society will move from the fourth to the third floor of the Chamber of Commerce building where it will have quarters 40% larger. The new quarters will have provisions for an assembly room which will be used also for noonday lunches.

The second week in May was "Membership Week" during which time over three hundred new names were added to the roll of the society so that its membership now totals 1,130. Preparatory to the membership week a list was prepared of all in Cleveland having necessary qualifications to become members in one of the three grades, Active, Associate and Junior. This classified list contained about 1,800 names. Adding to that the number already in the society, it is seen that there are 2,600 eligibles in the city.

American Society of Civil Engineers—The Forty-Eighth Annual Convention will be held in Pittsburgh June 27 to 30 at the William Penn Hotel. Following precedent the summer meeting will be devoted in about equal proportions to business, golf, social events and excursions. The opening meeting will be held Tuesday, June 27 at 10 a.m., at the William Penn Hotel. Mayor Armstrong, of Pittsburgh and John A. Brashear, Past President of the American Society of Mechanical Engineers, will welcome the convention. The annual address of the President will be given by Clemens Herschel. In the afternoon session results of the vote on change of headquarters will be announced and committee reports received. There is to be a dinner-dance that night. On Wednesday, an all-day excursion will be taken, by rail up the Monongahela to Lock No. 4 and returning on the Steamer "Sunshine" past the Monongahela industrial district, of the Pittsburgh Harbor. Thursday will be devoted to visits to Pittsburgh industries, under guidance of members of the local committee. The following plants will be visited: American Bridge Co., Ambridge Plant; Baltimore and Ohio R.R., 33rd St. improvement; Duquesne and Edgar Thompson Steel Works of the U. S. Steel Corporation; H. J. Heinz Co.; The McClintic-Marshall Construction Co., Rankin Plant; National Tube Co., McKeesport; Pennsylvania Rubber Co., Jeannette; Pittsburgh Water-Filtration Plant, Aspinwall; Universal Portland Cement Co., Universal; the United States Bureau of Mines and United States Bureau of Standards. Members will be asked to register their selection from among these trips. Thursday evening the President's reception and ball will be held. Friday is to be devoted to an all-day golf tournament at the Oakmont Country Club, with an automobile trip for those not taking part. The Engineers Society of Western Pennsylvania will be hosts at a smoker in the evening.

Appliances and Materials

Forged Steel Insulator Pins

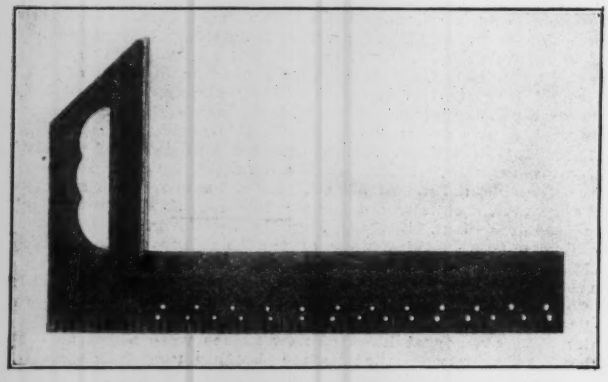
Two lines of forged-steel (open hearth) pins for transmission-line petticoat insulator pins are being made by Hubbard & Co., of Pittsburgh. One line is designed to carry a maximum of 3,000 lb. pull at the line-wire groove of the insulator; the other line is for 1,500 lb. Under such a load the pin deflection will not be more than 10° from the vertical. The bending takes place near the pin base so that the inner shell of the insulator does not break against the pin, as in the case of a cast-iron pin.

It is claimed that these pins eliminate the internal breakdown of insulators under electrical stress. A sheet-steel thimble, only $^3/_{64}$ in. thick, is cemented into the insulator, and this fits the pin loosely until a solid seat is secured by bringing up against a cork cushion in the top. This construction takes up the stress from pin expansion in the cork cushion; the small amount of metal in the thimble does not give sufficient stress in the porcelain to affect its properties, as has been thought to be the case with cast-iron thimbles.

* * *

Hole Templet for Belt Lacing

A templet square for correctly spacing holes for belt lacing, where the twisted round or wire belt lacing is used in making joints, is manufactured by the George W. Southwick



BELT-LACING TEMPLET

Co., Stamford, Conn. It is claimed to be a time saver in addition to insuring proper spacing.

* * *

Natural-Zeolite Water Softener

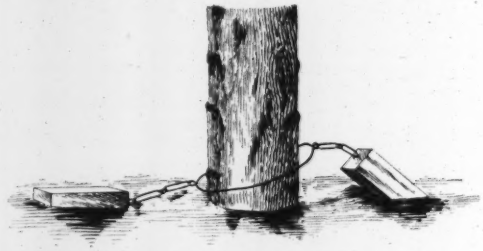
Under the trade name "Refinite," a natural mineral, zeolite, is being sold by the Des Moines Refining Co., 109 East Walnut St., Des Moines, for use in water-softening filters. The history of this development is unique. A mineral was found by a prospector in the Black Hills of Dakota, which was thought to contain fuller's earth; a sample was sent to a chemist in Des Moines, who identified it as bearing an aluminum-sodium silicate—one of the family of what are termed in geology "zeolites," having the property of easily exchanging its sodium for water-borne calcium. The chemist called in the engineers of the local water company to investigate the practicability of using the mineral in water-softening. Further studies were made by Alvord and Burdick, of Chicago, and Edward Bartow, of the University of Illinois. Finally the Des Moines Refining Co. was formed to develop the process commercially.

The use of the material is similar to that of artificial zeolites which have been found successful. The mineral is placed in a vertical cylindrical tank of such dimensions that a slow flow is secured from the top inlet to the bottom outlet. When the sodium of the mineral has been all exchanged for calcium and magnesium from the water which has been softened, a salt solution is run through and a reverse exchange effected, restoring the mineral to its original condition.

* * *

Pile Protection Against Wood Borers

The Common Sense Pile Protector Co., of Long Beach, Calif., has put on the market a novel device for keeping tidewater piles free from teredo, limnoria or other wood borers. The protector consists of a jointed metal ring or endless chain suspended about 16 in. below water level by floats attached at separated points. The protection is afforded by the combined scraping action of the wire, which has a diameter about 1 in. greater than the largest diameter of the pile, and the pounding action of the floats against the pile. These floats are so attached that they revolve on arms, rubbing and continuously striking the pile and at the same time this revolution magnifies the scraping action of the wire. A single protector will keep clean a pile from two feet below low water to high water, and a system of multiple rings is available for complete protection from high water to ocean bottom.



A SIMPLE WAY TO DESTROY TEREDOS

The protector is not sold or leased but the company contracts to give the service of protection at a yearly rate per pile. It is stated that installations on the Pacific Coast for the last four years have proved successful.

* * *

Double-Hopper Motor Dump-Wagon

The dump-wagon shown herewith consists of a pair of Lee side-dump hoppers mounted on the platform of a Knox tractor, and is intended specially for handling material on road construction. With the side-dump, material can be dumped on either side of the road, and leave a roadway through the middle of the work. As the turning radius is only 15½ ft. it can be handled and reversed very readily. With the two hoppers, material can be delivered at different points, or different materials can be carried (such as sand and stone for concrete). The hoppers are of 7½ tons capacity each, and are strengthened by interior diaphragms at the bottom. They roll on inclined paths, being kept in position by guide channels on the sides, while chains hold them at the end of their travel. As soon as the latch is released, a sharp pull on a lever is sufficient to start the hopper, whose shape is such that it gives a rapid discharge and throws the material clear of the wheels. The hoppers are of a type designed originally (in smaller sizes) for use in unloading stone, gravel, coal, etc., from railway cars. For this work they stand on steel A-frames, and are loaded by shovelers in the cars ready for

DOUBLE-HOPPER DUMP TRAILER AND TRACTOR

discharging into carts or dump wagons. They are built by the Lee Loader Co., of Chicago, which has adapted them for use on motor trucks and trailers, while the Chicago street railways have utilized them for dump cars. The motor dump-wagon illustrated was developed through the Chicago office of the Knox Motors Associates, of Springfield, Mass.

The tractor which is pivoted to the forward end of the platform, has a 40-hp. motor and can handle a 30-ton load on smooth roads. A test load of 15 tons of crushed stone was hauled over rough cobblestone paving at 16 mi. per hr., although such a speed is much too high for regular service. The first hopper being dumped, the tractor moved forward 5 ft. and dumped the second load on top of the first pile.